ASSESSING THE BOND QUALITY OF PRE-STRESSING STRANDS WITH SCC USING NASP TEST

By

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CHAPTER I

INTRODUCTION

1.1 BACKGROUND

"Necessity is the mother of invention". It was 1983, which showed the concrete industry a new dimension of research and advancements. Lack of skilled workers to construct durable concrete structures was an industry wide problem in Japan. To overcome the issue, Professor Hajime Okamura and Ouchi (February 1986, University of Tokyo, Japan) advocated a concrete that would consolidate under its own weight without the requirement of additional vibration at its plastic stage. The roots of the search ended with the development of a highly flow-able and stable concrete. This concrete can spread readily into place through intricate reinforcement configurations without any consolidation, reducing time, cost of construction, enhancing structural performance and durability; leading to what we popularly known today as – "**SELF CONSOLIDATING CONCRETE**".

Since 1983, several European countries like Netherlands, Sweden, United Kingdom and others have successfully implemented SCC into a wide gamut of infrastructure projects ranging from pre-stressed bridges, earth retention walls, to concrete box tunnels. However, in the United States it was first the concrete admixture manufacturers who introduced SCC into the precast and cast-in place applications. Since then, transportation agencies of New York, Nebraska, Virginia and other States had reaped its benefits by successfully implementing it in a substantial way.

Some of the highly successful projects include concrete bridge beams on the Brooklyn-Queens Expressway from 61st Street to Broadway in New York City. SCC is also being used in the reconstruction of the East Tremont Avenue Bridge over the Cross Bronx Expressway and a substantial portion of the precast substructure components for the replacement of the Roslyn Viaduct bridge, outside of New York City.

The Virginia Department of Transportation (VDOT) has used SCC for precast, pre-stressed, and cast-in-place applications. In 2001, SCC was used for precast components of an arch bridge near Fredericksburg. In 2005, VDOT used eight pre-stressed SCC beams in one span of the new Route 33 over the Pamunkey River Bridge near Richmond.

Figure.1.1: The skyline bridge in Omaha, NE, features a full width bridge deck made with SCC (FHWA, FOCUS, November 2005)

In Nebraska, the Department of Roads is using SCC for applications such as long-span and short-span bridge girders, pilings, and temporary Jersey barriers.

Researchers from the Federal Highway Administration's (FHWA), National Cooperative Highway Research Program (NCHRP), Advanced Cement Based Materials(ACBM) and others are working particularly in the development of mix designs, characterization of its rheology, mechanical properties and its in-situ properties.

In spite of all the progress made on the material characterization of SCC, very limited information exists on the construction and design specifications to guide industry, designers and the highway transportation authorities. Thus, if this trend continues, it is very likely that that even the most extensive and detailed material characterization efforts on SCC will not address the concerns of the state DOT's, whose ultimate interest lies on the structural performance and durability of the built structure.

1.2 STATEMENT OF PROBLEM:

Several different approaches were used to develop SCC. According to (Sakata et.al 1995, Bouzoubaa and Lachemi 2001, Lachemi et.al.,2003 a) the self consolidating property can be achieved by significantly increasing the amount of fine materials like fly-ash or limestone filler without changing the water content compared to common concrete. Another, approach consists of incorporating a Viscosity Modifying Admixture (VMA) to enhance stability (Sari et.al, 1999; Lachemi et.al, 2003a).

The Viscosity Modifying Admixtures are water soluble polymers that increase the viscosity of the mixing water and enhance the ability of the cement paste to retain its constituents in suspension. VMA's are used to enhance the stability of the SCC (Rols et.al.1999; Khayat and Guizani ,1997 ,Lachemi et.al 2003a). The use of VMA along with the adequate concentration of the superplasticizer can enhance high deformability and adequate workability leading to good resistance to segregation. According to (Sakata et.al., 1996) the use of VMA was proven to be effective but they tend to be costly and increase the overall price of the SCC. Thus, a cost effective VMA could significantly reduce the price of SCC which could encourage the use of the concrete in the new construction.

The various mix designs that gave SCC its unique fresh property advantages differed significantly from one another. This had raised concerns regarding the material and structural performance issues. Among them are the material properties like the creep and shrinkage; structural issues such as the pre-stress loss and bond; durability issues such as freeze-thaw behavior. These concerns have limited the acceptance of SCC in the United States, despite of its increased use in Japan, Canada and Europe.

Researchers have been debating the issue of bond for the pre-stressed members since the past six decades. There are considerable works that have been done regarding the development of the better understanding of bond and its relationship with the transfer and development length for the conventionally consolidated concrete. The conventional concrete which is well developed and ideal showed deviated results related to the bond issues, mainly in the equation of transfer length. Hence, with the absence of specific design codes and guidelines for SCC, the study of bond in SCC has become inevitable.

1.3 RESEARCH OBJECTIVES:

The unusual characteristic associated with SCC does not allow us to use the conventional standardized testing to monitor the quality of the concrete correctly after it is being placed. Irrespective of the design of concrete mix for SCC the requirement of standardized testing needs to address three specific properties of SCC:

1. Filling Ability.
2. Passing Ability.
3. Segregation Resistance.

The current research focuses on identifying the testing procedures that can assure fresh concrete properties which will sufficiently fill the forms, flow around reinforcement and other congestions, consolidate under its own weight and resist segregation. Moreover, the bond performance of SCC is investigated by the NASP bond test so that we get a future guidance on the construction and design of pre-stressed /precast bridge elements when using Self Consolidating Concrete.

The three main objectives of this study are:

1) To investigate the material properties of three types of SCC mixes through small scale tests.
2) To experimentally investigate the bond performance of SCC using standard NASP bond test.
3) To perform statistical analysis and comparing the pull out values with the conventional mortar mix.

The knowledge gained from this research and resulting improvements to the state of art will definitely assist the engineers in the safe design of the precast, pre-stressed concrete bridges using SCC.

1.4 SELF COMPACTING CONCRETE (SCC) V/S NORMAL CONCRETE.

The very unique property of the self consolidating concrete and its tailor able nature to suit specific project needs has allowed it to be classified as a kind of high –performance concrete in its plastic state. It offers many advantages for the precast/pre-stressed industry, among them are:

1) Its ease of placement with need for no mechanical vibration resulting in the savings in the placement costs.

2) When it is placed in a form, its motion may be either a creeping movement or a rapid flow. This style of flow creates a much improved surface finish between the form and the concrete, over the conventional concrete.
3) Its ability to flow through the complicated reinforcing sections only by the energy of its own weight without creating blockage gave it an edge over the normal concrete.
4) Demanding form configurations, irregular shapes, thin heavily reinforced elements SCC have shown considerably impressive results when compared to conventional concrete.
5) The improved consolidation around the reinforcement and bond with the reinforcement.
6) Improved Pump ability as compared to Normal Concrete.
7) Improved uniformity of in place concrete by eliminating variable operator-related effort of consolidation.
8) Labor Savings.
9) Shorter construction periods and resulting cost savings.
10) Safety hazards are also reduced in the plant as the use of SCC minimizes the need for workmen to walk on the top of the form thereby eliminating the cords and hoses associated with the concrete vibrators. In all it results in a significant reductions in accidents when SCC has been introduced into the pre-cast production activities.

To summarize in order to completely utilize the advantages SCC offers, especially to the pre-cast/pre-stressed industry, bond parameters need to be carefully evaluated. Thus, considering the extensive mix designs of SCC available, the current work focuses on effectively bounding the hardened properties by proper mix designs.

CHAPTER II

ELEMENTS OF SELF-CONSOLIDATING CONCRETE

"Self-Compacting Concrete – the New Wave" as it had been correctly stated by Millar.D (2000). To substantiate the ongoing zeal of successfully implementing self consolidating concrete with pre-stressing strands understanding the issue of bond is in-evident. Thus, a literature review was carried on the various aspects of pre-stress bond issues and the properties of the self –consolidating concrete. This chapter provides the pertinent work related to the research and subdivided into four major sections:

(1) SCC material technology.

(2) Issue of bond in pre-stressed concrete.

(3) Concluding remarks.

2.1 SCC MATERIAL DEFINTIONS :

According to Khayat , Hu and Monty(2000) ; SCC is a highly flowable , yet stable concrete that can spread readily into place and fill the formwork without any consolidation and without undergoing significant separation. According to Okamura and Ouchi (1986), the three stages of the concrete can be classified as: fresh (self-compactable), early-age (avoidance of initial defects) and hardened (protection against external factors). Due to these inherent properties, SCC can be termed as a "high-performance concrete" in the plastic state. As it had been stated in the previous chapter, SCC has many advantages over normal production concrete, that are used in the precast/pre-stressed concrete plants. Owing to the features like easily producing horizontal and vertical components with block outs and crowded reinforcing, better architectural and textured surface finishes, SCC requires a higher level of quality control. In addition, a greater awareness of aggregate gradation, mix water control and the use of highly advanced high range water reducing admixtures or the viscosity modifiers is to be exhibited. Several terms are used in the discussions of the material properties of self –consolidating concrete. This section states and explains the definitions of the most significant terms used in the relevant discussions. The PCI is “Pre-stressed Concrete Institute”.

Admixtures:

According to Portland Cement Association, a material other than water , aggregates, hydraulic cement, and fiber reinforcement , used as an ingredient in concrete or mortar, and added to the batch immediately before or during its mixing to modify the properties of the fresh or hardened concrete.

Aggregate Aspect ratio (as per PCI guidelines):

The ratio of the length to the width of individual pieces of coarse aggregate is known as the aggregate aspect ratio. This ratio sometimes affects the characteristics of SCC.

Aggregate Blocking (as per PCI guidelines):

The situation in which coarse aggregate particles jam between reinforcing steel bars or other obstacles within the form and prevent free flow of SCC.

Binder (as per PCI guidelines):

The combined cement and hydraulic powder added in a SCC mix is known as the binder. Cementing materials, either hydrated cements or lime and reactive siliceous materials – are used to form the matrix in SCC.

Bingham Fluid (as per PCI guidelines):

A fluid characteristic by a non-null yield stress and a constant viscosity regardless of the flow rate is referred to as Bingham Fluid. It is explained in detail in the yield stress and thixotropy section later.

A detailed analogy showing the difference between the this and the Newtonian fluid is discussed later.

Blocking (as per PCI guidelines):

The condition in which pieces of coarse aggregates combine to form elements large enough to obstruct the flow of the fresh concrete between the reinforcing steel or other obstructions in the concrete formwork is referred to as blocking. This property is of increased importance in SCC because of the absence of vibration energy to dislodge these blockages.

Cohesiveness (as per PCI guidelines):

The tendency of the SCC constituent materials to stick together, resulting in resistance to segregation, settlement and bleeding is referred to as the cohesiveness of the mix.

Compactibility (as per PCI guidelines):

The ability of the SCC mix to form a dense compact mass without the requirement for input of external energy (vibration) is known as its compactibility..

Consolidation (as per PCI guidelines):

The process of inducing a closer arrangement of the solid particles in freshly mixed concrete or mortar, during placement by the reduction of voids; usually in non-SCC by vibration, configuration, rodding , tamping or some combinations of these actions is referred as consolidation. In SCC, consolidation is by gravity flow of the material.

Stability (as per PCI guidelines):

This term is also known as the segregation resistance. The ability of SCC to remain in homogeneous in composition by resisting actions, which make the constituents separate from the mass during transport, placement and subsequent to placement is referred to its stability.

This is generally divided as the static stability and dynamic stability.

Static Stability (as per PCI guidelines):

This is also referred to as static segregation resistance. The characteristics of a fresh SCC mixtures that ensures uniform distribution of all solid particles and air voids once all placement operations are complete and until the onset of setting, without excessive settlement or bleeding is its static stability.

Dynamic Stability (as per PCI guidelines):

The characteristic of a fresh SCC mixture that ensures uniform distribution of all solid particles and air voids as the SCC is being transported and placed.

Filling Ability (as per PCI guidelines):

The ability of SCC to flow under its own weight (without vibration) into and fill completely all spaces within intricate formwork, containing obstacles, such as reinforcement is known as the filling ability of the mix.

Flowing Ability (as per PCI guidelines):

The ability of a fresh concrete to flow in a confined or unconfined form of any shape, reinforced or not, under gravity and external forces, assuming the shape of the container is referred to as the flowing ability of the mix.

Passing Ability (as per PCI guidelines):

The ability of SCC to flow through the openings approaching the size of the mix coarse aggregate, such as the spaces between steel reinforcing bars without segregation or aggregate blocking is known as the passing ability of the mix.

Plastic Viscosity (as per PCI guidelines):

The condition of freshly mixed concrete such that deformation will be sustained continuously in any direction without rupture is referred as its plastic viscosity. This can also be stated as the measurement of a material's resistance to increase in its rate of flow with increasing application of forcing energy. This is explained in detail in the later sections.

Rheological Properties (as per PCI guidelines):

The properties dealing with the deformation and the flow of the fluid fresh SCC mixture is known as its rheological properties.

Thixotropic Behavior (as per PCI guidelines):

The property of a material that will allow it to exhibit a low viscosity while being mechanically agitated , but stiffen after a short period at rest is referred as its thixotropic behavior.

Yield Stress (as per PCI guidelines):

One of the rheological constants of fresh concrete, fresh mortar and fresh paste when they are regarded as Bingham fluids. The minimum stress required to make the concrete flow is referred as its yield stress.

In order to completely understand the material properties that will result in the development of self – consolidating concrete the concept of "fluidity" plays an important role. However, to quantify this , a co-rrelationship must be established between the shear stress with the shear rate of the mix. Thus, to achieve a highly mobile concrete, a low yield stress is required and for a high resistance to segregation, a highly viscous material is important. Water can be added to decrease the yield stress but this addition also lowers the viscosity. Addition of a super-plasticizer will also lower the yield stress but will only lower the viscosity slightly. The viscosity of the mix can be increased by changes in the mix constituents or the addition of a viscosity modifier, but this will increase the yield stress of the paste. To summarize a plot of shear stress V/s shear rate of SCC can be delineated as follows:

$$\tau = \tau o + \mu\gamma$$ **(Newmann, Choo 2003)**

where **τ** - shear stress applied to material,
τo - yield stress
μ - plastic viscosity
γ - rate of shear

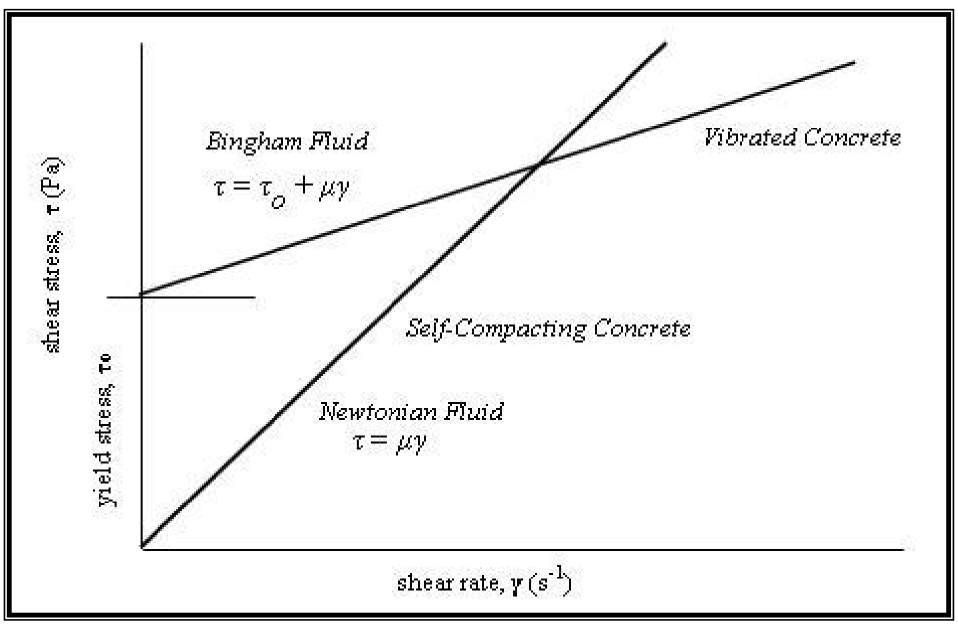


Figure 2.1: A comparison between conventional concrete and SCC, (J. Newman, Ban Seng Choo,2003)

A detailed study of the material properties and its constituents is done in order to validate linear relationship and the properties of the Newtonian liquid. The rheogram showed above differentiates between the between types of the fluids. The Newtonian fluid rheogram is a straight line passing through the origin. The slope of the line is the viscosity and the intercept on the x-axis is the yield stress. For a Newtonian fluid , the viscosity is independent of the shear rate and may depend only on the temperature and perhaps pressure. All the fluids for which the viscosity varies with the shear rate are the non-Newtonian fluids. However, for the non-Newtonian fluids the viscosity defined as the shear stress to the shear rate (often called the apparent viscosity) is emphasized to distinguish from the Newtonian behavior. Thus, for the Non-Newtonian fluids a finite stress is required before the continuous deformation occurs, however SCC due its inherent filler contents does not require any of these.

2.1.1 Cement and Fine Fillers:

The majority of the research on SCC has been carried out using Portland Cement (PC) as the binder. In developing the mix , the total fines content of the mix is balanced against aggregate size and grading , however the fines content is much higher than the conventional concrete in order to attain stability. According to the Domone and Chai (1996), Shindoh(1996) and Nishibayashi (1996), addition of fine particles binder like PFA, GGBS enhanced the workability of the fresh normal concrete mixes and beneficial effects of these materials resulted in the reduction of PC contents and lower heat of hydration(Khayat 2000; Henderson,2000). According to Petersson (1996), Khayat (2000); ultra fine binders like CSF maintained the desired workability by increasing the superplasticizer dosage. However, the most widespread material is the limestone filler powder due its economical feasibility. According to Domone and Chai(1997); the limestone filler helped to maintain the stability in a high workability mix although it will not contribute significantly to the compressive strength development of SCC. To summarize, with the various combinations of binders and fillers , the packing of materials strongly influences the plastic viscosity and yield shear stress.

2.1.2 Fine and Coarse Aggregates:

The fine aggregate in SCC plays an important role in the workability and the stability of the mix. The total fines content of the mix is a function of both the binder (and filler) content and the fine aggregate content with the grading of the fine aggregate being particularly important. Thus, the sands with the fineness moduli of between 2.4 to 2.6 have been used in producing SCC. However, finer sands are recommended to ensure satisfactory segregation resistance. SCC has been made with both gravels and crushed rock as coarse aggregate. The maximum aggregate size is mostly 20mm but smaller aggregate size upto 10mm have also been used.

2.1.3 Admixtures:

To achieve an optimum balance between the levels of consistency (workability) required from SCC while maintaining stability of the mix has led to the use of a number of admixtures within the concrete. The advances in the admixture technology played a vital role in the development of SCC. Modern super-plasticizers (based on poly-carboxylic ethers) promote good workability retention and can be added at any stage of the batching cycle.

The poly-carboxylic based super-plasticizers achieve this with a mechanism of electrostatic repulsion in combination with the steric hindrance. Most of the viscosity modifying admixtures is high molecular weight polymers with a high affinity to water. By interaction of the functional groups of the molecules of water and the surface of the fines , VMA's build a three dimensional structure in the liquid phase of the mix to increase the viscosity and the yield point of the paste. The strength of the three dimensional structure affects the extent to which the yield point is increased. Some VMA's are based on the inorganic materials such as colloidal silica which is amorphous with small insoluble , non-diffusible particles , larger than molecules but small enough to remain suspended in water without settling. By ionic interaction of silica and calcium from the cement a three dimensional gel is formed which increases the viscosity and the yield point of the paste. This there dimensional structure /gel contributes to the rheology of the mix , improving the uniform distribution and suspension of the aggregate particles and so reducing any tendency of bleeding, segregation and settlement. Most of these are supplied as either as a powder blend or are dispersed in liquid, the dosage depends on the application but typically ranges from 0.1-1.5% by the weight of the cement.

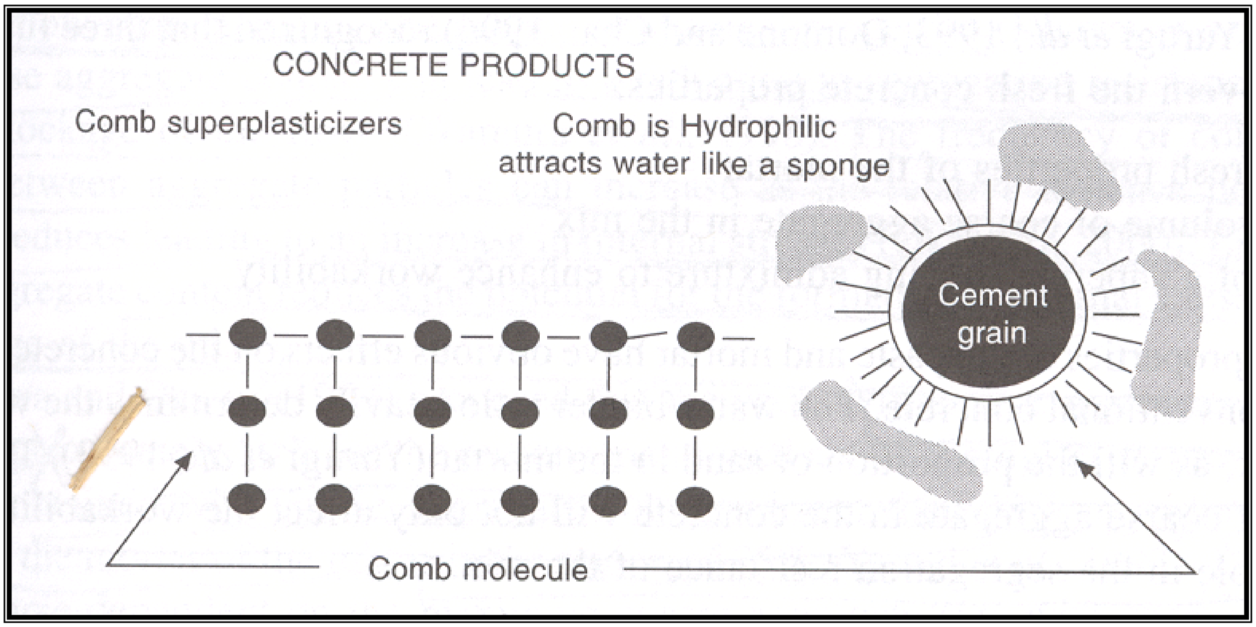


Figure 2.3: Mechanism of poly-carboxylic super-plasticizers (Grace Construction Chemicals, 2006).

The VMA are high molecular weight soluble polymers, which in aqueous medium have increased viscosity because of their interaction with water. According to Okamura (2000), these admixtures are effective in stabilizing the rheology of the fresh concrete and preventing the segregation of the coarse aggregate from the other mix constituents.

2.1.4 CONCEPT OF RHEOLOGY:

Fresh SCC behavior can not be fully comprehended without understanding its rheology. Rheology in the broad sense is the science of flow and deformation of matter. The placing, spreading, pumping and compaction of any concrete depends on rheology. Using the science rheology it is becoming possible to predict fresh properties, select materials and model processes to achieve the required performance. Rheology is now seriously considered by concrete users, rather than being seen as an area of specialized domain of cement science. The basic key rheological parameters are yield stress and plastic viscosity as explained in the diagram earlier, in Figure 2.1. Fresh concrete can be seen as a fluid, providing that a certain degree of flow can be achieved and the concrete maintains its homogeneity. Self-compacting concrete is a two phase (solid and liquid) particle suspension and is very fluid. The challenge than is to maintain the flow-ability of the suspension and to avoid the segregation of the phases. The main mechanism to control the flow-ability and stability of SCC is related to the surface chemistry. Thus, development of SCC has been strongly dependant on surface active admixtures as well as particle packing properties. However, fresh SCC is a combination of solid phase (F.A and C.A) and the liquid phase (cement, air, admixtures). Thus the factor converges into two concepts: 1.) the aggregate blocking model (solid phase) and 2.) the paste model (liquid phase). According to Shah, Akkaya, Bui (2002) the paste model was a function of minimum apparent viscosity, minimum flow and optimum flow- viscosity ratio. These criteria are related to the average aggregate diameter, aggregate spacing which is directly proportional to the paste volume (or aggregate volume), average diameter and void content of aggregates.From these mechanical models, we conclude that there are three distinct and different features of a SCC paste. We need to understand the basic concepts involved in this:

- Paste Viscosity.
- Yield stress and thixotropy of complex fluids.
- Structure (Inter-particle interaction).

The balance between the yield point and the paste viscosity is the key to obtain the appropriate concrete rheology. The viscosity modifying admixtures (VMA) change the rheological properties of the concrete by increasing the plastic viscosity, but usually causes only a small increase in the yield point. However, the admixtures which decrease the yield point are the plasticizers and are often used in the conjunction with the VMA to optimize the yield point.

These three mechanisms combine together to develop what is today called "Self-Consolidating Concrete". However, the inter-particle interaction in the matrix is governed by the yield-stress, viscosity and density of the cement paste matrix. The morphology of the particles are not enlisted as the particle size distribution and inter-particle spacing are components and contributor to both structure and plastic viscosity of the mix. However, in order to completely grasp the concepts of this advanced concrete technology, SCC terms should be understood from the fluid mechanics point of view.

2.1.5 Paste Viscosity:

The fluid continues to deform as long as the force is applied, unlike a solid which would undergo only a finite deformation. Within the fluid, a linear velocity profile is established, due to the no-slip condition, the fluid bounding the lower the plate moves at the plate velocity. A basic concept showing the true meaning of "viscosity" is displayed here. Whenever any fluid is subjected to a shearing force, one layer of molecules glides one over to its layer underneath. The molecules which are in contact with the shearing surface moves along with while the rest moves at a relatively lesser pace or zero at the stationary surface. This generates a “velocity gradient” between the two layers. This is shown in the diagram below:

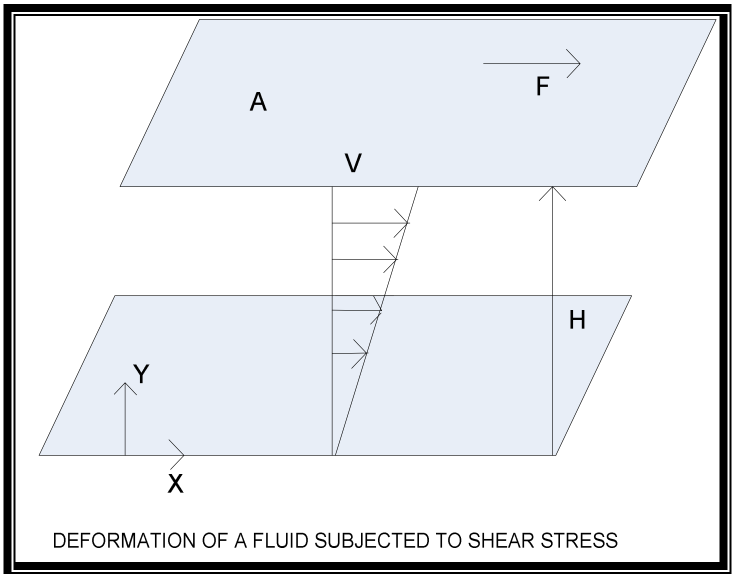


Figure 2.4: Deformation of a fluid subjected to shear stress.(Perry, Handbook 2003)

The velocity gradient is the shear rate for this flow. The ratio of shear stress to the shear rate is the viscosity. It measures the materials resistance to flow.

However, in SCC the viscosity of the paste depends on the aggregate volume (paste volume), particle size distribution of the fine and coarse aggregates, aggregate shape, fine – coarse aggregate ratio, characteristics of the aggregate surfaces , difference of density between the aggregate and paste, and the usage of admixtures. The cement paste is shear thinning and the viscosity decreases with the increasing shear rate. The shear thinning fluids are those for which the slope of the rheogram, as discussed in the previous section decreases with the increasing shear rate. The shear thing fluids without the yield stresses obey a power law model for a range of shear rates: **$\tau = K^*\gamma$^n (Herschel-Bulkley)**

Where K is the consistency index or power law coefficient, n Is the power law exponent. For shear thinning fluids, n<1. The shear thinning power law fluid (like SCC) to certain extent with the yield stresses are the “ Herschel-Bulkley fluids”, which is explained in detail in the yield stress section. According to Saak, Jennings,Shah (2001), the fine and the coarse aggregates in the SCC produces a continuous particle size distribution. The fine aggregate particles will segregate at a lower paste yield stress and viscosity than the larger coarse aggregates. They found that the fine aggregates place an additional upward force on the coarse aggregate, somehow resisting the separation of the matrix constituents. Also , the aggregates are assumed to be packed to a maximum particle density, giving a very well-defined geometry of the system.

According to Shah, Akkaya and Bui (2002), the void content , average diameter and the volume of the aggregates affect the spacing between the surfaces of aggregate particles. The aggregate dimension has an effect on the segregation resistance and deformability of SCC.

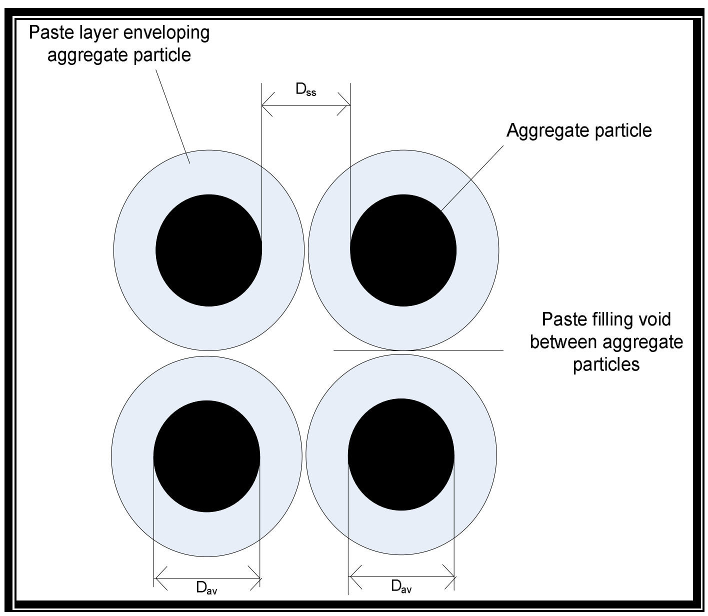


Figure 2.5:Showing the effect of aggregate spacing and aggregate diameter on SCC (Bui, Akaya, Shah 2002)

They correlated that the average aggregate spacing Dss (as shown) and average aggregate diameter Dav (as shown) with the aggregate properties and content. They suggested that the paste volume must be high enough to fill the voids between the aggregate particles and create a layer enveloping the aggregate particles to achieve high deformability and good segregation resistance. The average aggregate spacing Dss is defined as an average distance between the surfaces of the aggregate particles or as twice the thickness of the paste layer around an aggregate particle. They concluded that the minimum viscosity of the paste depends not only on the aggregate spacing but also on the average aggregate diameter. For certain aggregate spacing , the lower value of aggregate diameter requires a higher viscosity of the paste. This is due to the fact that, for the same aggregate spacing, SCC with a smaller value of Dav had a smaller volume of coarse and total aggregates. The aggregate combination which leads to the smaller average aggregate diameter, had a larger proportion of fine particles, which increases the total volume of the powder, thereby reducing the volume of the coarse aggregate. The smaller volume of aggregate resulted in the less friction risk among the aggregate particles and this requires a higher paste viscosity to hold the aggregate particles together. They also showed that the paste flow and the paste viscosity cannot be varied independently to assure SCC. The ratio of the paste flow and the paste viscosity were examined.

The optimum flow-viscosity ratio was related to the aggregate spacing , higher aggregate spacing required a lower flow-viscosity ratio. Higher aggregate spacing demanded a lower flow and higher viscosity of the paste in order to obtain sufficient deformability and segregation resistance. However, a lower aggregate diameter generally required a lower optimum flow-viscosity ratio.

2.1.6 Yield Stress and Thixotropy of complex fluids:

When the solids or very viscous liquids are subjected to stresses that are high, but not so high that they cause fracture, a process known as the yield or the plastic deformation occurs.

Under an increasing shear strain, each row of atoms is displaced from its equilibrium position with respect to the neighboring row. Below a critical strain, if the stress is removed, the atoms spring back to their original positions. However since the arrangements of the atoms are spatially periodic, if the deformation continues, each row of atoms will eventually find itself back in the registry with its neighboring rows, with each atom simply displaced by one inter-atomic distance from its original position relative to the layer below it. The strain energy is therefore a periodic function of strain, oscillating between zero and a maximum value. The stress, which is the derivative of the energy with respect to the strain, therefore oscillates about zero, with a positive maximum value and negative minimum. Beyond yield strain, after reaching a maximum stress there will be an instability whereby some layers slide forward and others spring backwards to their initial positions with respect to the underlying layer with the result that all the layers find themselves in registry with their neighbors, even though the strain is very small. Thus the yielding involves slippage between some pairs of neighboring layers, with no relative movement at all between other pairs. Hence, the strain is not homogeneously distributed throughout the sample, but is localized to certain slip planes.

In order to completely understand the structure and the rheology of complex fluids like the SCC we need to understand the Herschel-Bulkley(H-B) model. A typical model as suggested by "Herschel –Bulkley (H-B)" satisfying this is:

$$\sigma = \sigma y + a*\ddot{Y}^\wedge n. \text{ (Herschel – Bulkey)}$$

where a and n are the positive constants,

σ is the applied shear stress and

σ_y is the yield stress is referred to as H-B model.

If $\sigma_y = 0$, $n=1$ the H-B fluid degenerates to a Newtonian fluid with a viscosity a. However, when subjected to a stress, the response of an H-B fluid (like SCC) is a slow shear flow provided the stress is slightly above the yield point. Theoretically, the yield stress is defined to be stress at which the fluid starts moving i.e, when viscosity changes. In the H-B model this is σ_y. The non- Newtonian fluids for which a finite stress is required before the continuous deformation occurs are the yield stress materials. In addition the yield stress fluids are the fluids that can support their own weight to a certain extent i.e. , they can support shear stresses without flowing as opposed to Newtonian fluids. This means that a yield stress fluid on an inclined flow will not flow if the slope is below some critical angle, but will flow as soon as the angle becomes large enough. According to the H-B model, the material will just start flowing when an angle is reached for which the yield stress equals the gravitational force per unit area

$$\sigma_y = \rho * g * h \sin\theta,$$ (Moller, Bonn, Mewis 2006)

where, ρ the density of the material,

g the gravitational acceleration and

h the height of the deposited material.

Thus, the fluids like SCC where the handling, transport and placement are subjects of great concern the engineering design variables are properly proportioned in order to account for suitable “yield stress”. Another aspect of these kinds of fluid is the “shear localization”. However the H-B model asserts that all shear rates are possible and presupposes that the flow is always homogeneous at these shear rates, this generally fails as only a small region of the material actually moves and the rest remains static. This is due to the shear banding (localization) and wall slip. This means that there is a stress variation throughout the sample. Within the sheared region, the stress is higher than the yield stress and outside the region it is low. This shear localization happens below a critical shear rate. Below this critical shear rate, all the flow is localized in a region close to the shearing wall.

In addition, the thixotropic fluids are the fluids with a variable viscosity which reversibly decreases with time under high shear rates. Due to the reversibility, the viscosity increases in time at low or zero shear rates. However , thixotropy in simple terms can be defined as that the viscosity of fluids mainly caused by the microstructure of the interconnecting particles in the fluid which resists large rearrangements. When sheared, this structure is broken down in time and the viscosity consequently decreases in time. When left at rest, the microstructure slowly rebuilds itself and the viscosity increases. When the viscosity was measured as a function of time, for stresses smaller than a critical stress, the viscosity of the sample increases with time until the flow is halted altogether: the steady-state viscosity is infinite. On the other hand, for a stress only slightly above σc (critical stress), the viscosity decreases with time towards a (low) steady state value ηo from the Figure 2.6. The important point here is that the transition between these two states is discontinuous as a function of the stress. This gave two new concepts of "aging" and "shear rejuvenation".

Aging refers to the increase in the viscosity of the fluid with time at rest and under slow flow where the stresses are much smaller than the critical stress. For a stress that is slightly more than that required for aging, the microstructure in the fluid destructs, and the fluid starts to flow. There exists a critical stress that bounds a region of no flow for smaller stresses and a region of fast flow for higher stresses.

According to Shah, Ferron, Gregory, Sun (2007), very little information was gathered related to the thixotropy of the SCC mixtures. They developed a protocol consisted of hysteresis loops and energy methods to quantify the degree of structural rebuilding in the cement pastes. They took into account the stiffening due to the thixotropic rebuilding and irreversible structural changes by focusing on the rate in which the cement paste was able to regain its internal structure after shearing. In order to evaluate and compare the rate of rebuilding of different materials, a datum was established for the initial reference condition called as the ***equilibrium condition***. This was nothing but a relationship that was established between the shear stress and the time. Here at any given shear rate there was a condition in the microstructure in which available bonds for a given energy level were broken and the energy level was minimum. This state was called the ***equilibrium level***. They claimed that once the equilibrium state was achieved, the rate of the rebuilding was investigated by testing the specimen after different resting times. They performed different hysteresis loops for different resting times based on the initial equilibrium loop. This process was repeated with the new rest period being twice the amount of the previous rest period. The area on the rheogram plane between the up curve of each hysteresis loop and the corresponding equilibrium line was used to evaluate the rebuilding that occurred in the specimen. This area had the physical dimension of energy per unit time per unit of the volume and called the ***specific rebuilding energy (SRE)***. However the target fluidity was achieved by either varying the high range water reducing admixture dosage or by varying the water to cement ratio (w/c). They found that in order to increase the rate of structural build up, it was better to use high range water reducing admixtures rather than increasing the water content to obtain the fluidity level.

Co-relation between yield stress and thixotropy:

The concept of yield stress and thixotropy of the fluid originate from the same basic principles of aging and shear rejuvenation which dominates the structure of the fluid. To co-relate the interplay between flow, structure and viscosity, there exists a parameter which measures the local degree of interconnection of the microstructure. The viscosity increases with increasing this parameter which is the measure of the number of connections per unit volume. For an aging sytem at low or zero shear rate , this parameter increases while if the flow breaks down the structure , local interconnection in the microstructure decreases and reaches a steady state value at sufficiently high shear rates. To explain this in detail, a graph between the shear stress and shear rate is plotted. The curve Figure 2.6, below shows that for low shear rates the stress decreases with increasing shear rate, whereas for high shear rates it increases. This defines both a critical stress and a critical shear rate. For shear rates smaller than the critical shear rate the flow curve has a negative slope. This corresponds to a negative viscosity and signals unstable flows.

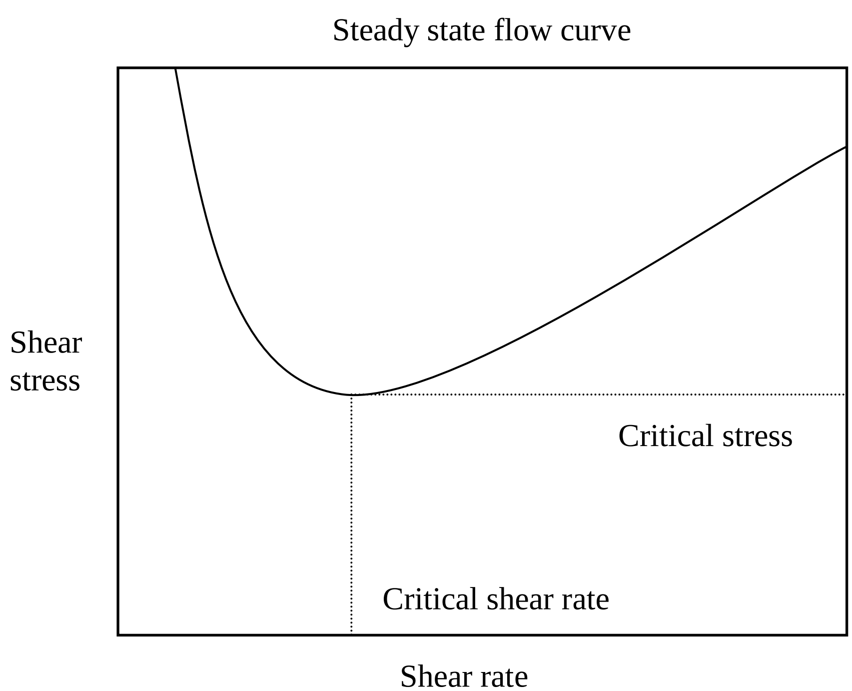


Figure 2.6 Relationship between Shear Stress and Shear Rate (Moller, Mewis and Bonn , 2006)

2.2 Structure:

The ability of the SCC to remain homogeneous in composition by resisting actions, which make the constituents separate from the mass during transport, placement and subsequent to placement-is its “segregation resistance” (PCI Interim Guidelines). The influence of the physical properties of the particles on the rheology of the cement paste suggested that the inter-particle separation (IPS) must be in conjunction with particle packing density. Particle packing density defined the amount of paste needed to fill the interstitial voids between the aggregate and minimum inter-particle distance referred to minimum thickness of the paste around each particle which will prevent the inter-particle friction, collision and subsequent blocking as explained in the section earlier. This is controlled by the particle size distribution of the aggregates and the volume percent of the binder which is required to fill the interstitial voids between the aggregates. According to Shah, Jennings, Saak (2003) in segregation control theory, the particles must remain suspended while the material is at rest, with only the minimum segregation occurring due to creep. It is also important that the particles move with matrix as a cohesive fluid during flow, avoiding both the static and dynamic segregation. The size (and density) of a particle can be estimated from the balance between the buoyant and gravitational forces acting on the aggregate and the restoring force due to the yield stress of the cement paste matrix as shown in the diagram. The gravitational and the buoyant forces are the functions of density of the aggregate, density of the matrix and the ***particle volume*** (Vp). Bonen and Shah (2005) found that the aggregate sedimentation and the segregation resistance was directly proportional to the viscosity of the mixture, mixture density, size and density of the aggregate and the flow ability of the mixture. They documented that in a SCC mix the coarse aggregates are in a well suspended mix. As shown in the Figure 2.7, during the horizontal flow the particle are subjected to mixture drag and vertical drag to keep the particles suspended in the mixture. They are proportional to the square of the velocity of mixture and aggregate shape. However, they found that at rest during static stability the gravitational force, frictional force and the buoyant force balance the aggregate.

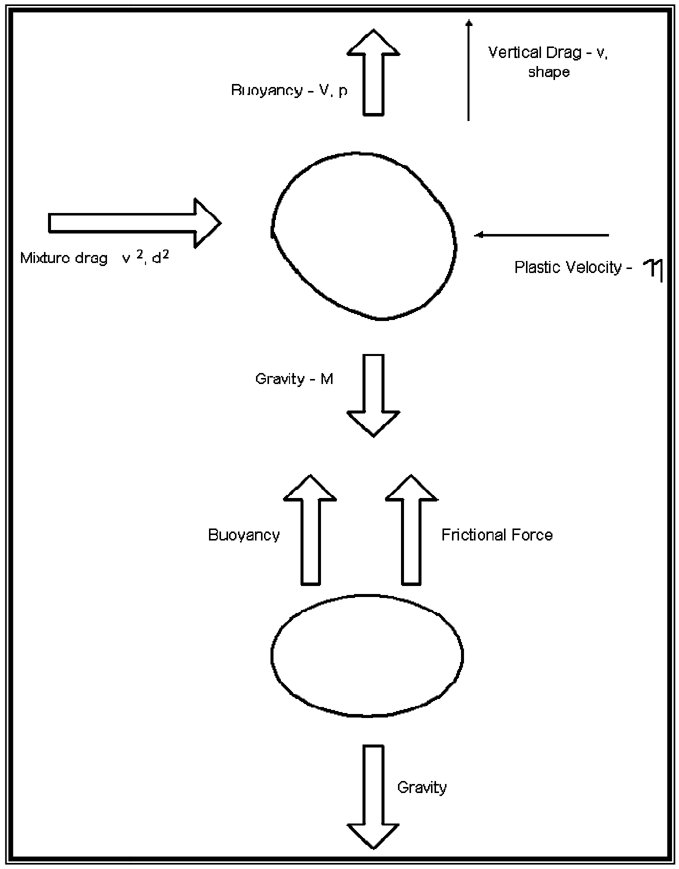


Figure 2.7: Forces acting on a suspended particle in a SCC mix.(Bonen, Shah 2005)

However, the restoring or the drag force exerted by the cement paste matrix is proportional to the yield stress, which is nothing, but the stress required to initiate the macroscopic flow. The density, yield stress and the viscosity of the cement paste matrix controls the segregation resistance for a given aggregate distribution in the concrete. The minimum yield stress and viscosity are dependent on the density difference between the paste aggregates; it should be optimized at a certain level in order to avoid both static and the dynamic segregation. However, their research kicked in a new concept of “Self –Flowing Zone”. The concrete will have its greatest fluidity at the lowest paste yield stress and viscosity where segregation is avoided. On the contrary, the segregation resistance is optimized at the highest yield stress and viscosity that still produces self-flowing materials. This created a critical range of the yield stress and viscosity where the segregation is minimized, yet the material is self-flowing. This segregation-resistant, high workability region is designated as "Self-Flowing Zone".

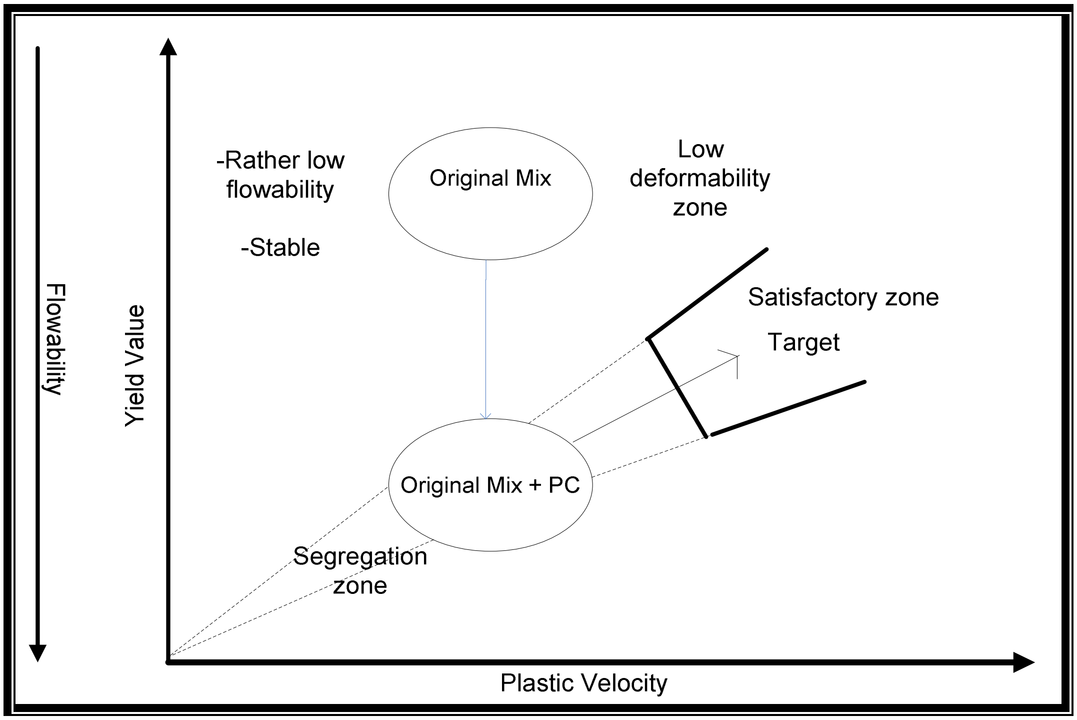


Figure 2.8: Concept of Self-Flowing Zone (Bonen, Shah 2005)

In addition, according to Shah, Saak, Jennings (2001) the yield stress of the matrix is exceeded during the placement, transportation and handling, thereby resulting the concrete to flow at a low shear rate. They found that if the density of the particle is greater than the density of the matrix , segregation or separation of the constituent particles will occur to a certain extent. If the viscosity of the matrix is high enough , the velocity of the falling particle will be slow resulting in no segregation. Thus during dynamic stage the drag force is given as:

$$\mathbf{Fdrag = Cd*\rho m*v^2/2*Ap}$$

Where C_d is the drag-coefficeint,

v is the terminal falling velocity of the particle,

A_p is the cross-sectional area of the particle submerged in the cement paste,

ρ_m is the density of the matrix.

To avoid segregation , the terminal velocity of the falling particle must be minimized. They found that the drag coefficient is proportional to the particles Reynolds number which is basically the ratio of the inertia to viscous force. In SCC, as the matrix is a suspension of the cement paste and aggregate, the terminal velocity is a function of the density difference between the cement paste matrix and aggregate.

2.3 SCC – A perfect solution for technology hungry people:

2.3.1 Need for SCC?

Compaction plays an important role in the development of hardened concrete properties. When certain properties are considered for performance of concrete structures it is assumed that concrete is well compacted and homogenous; the purpose of compaction hence is therefore to achieve the highest possible density. Vibration, which is still the most common way of compacting concrete, has the effect of fluid on the mortar component of the mix so that internal friction is reduced and closer packing of concrete aggregate takes place. But compaction via vibration is a discontinuous process resulting in hardened concrete with uneven compaction and therefore with different mechanical and durability properties. The avoidance of vibration was not the primary reason for the development of SCC, however; the starting point was a growing concern about difficulties of assuring the quality of complex concrete structures because of poor concrete compaction of in-situ. This led to increased construction costs and jeopardized long-term durability of structures. The only practical option was to eliminate the reliance of the concrete compaction on concrete workers and to replace it with the ability of concrete itself to guaranty the full filling of formwork, perfect compaction, and the full encapsulation of all reinforcing bars.

2.3.2 Fresh SCC

The functional requirements of fresh SCC are different from those of the CVC (conventionally vibrated concrete). SCC is a liquid particle suspension and exhibits very different properties in its plastic state. The following properties(filling ability, passing ability and segregation resistance) define the compliance with the self-compactibilty:

Filling Abililty:

The complete filling of formwork and encapsulating of reinforcement inserts and substantial horizontal and vertical flow of the concrete within the formwork while maintaining homogeneity is referred to as its filling ability. Filling ability is normally measured by either slump flow (Fig:2.9 a), or J-Ring (Fig.2.9 b) tests.

Figure 2.9 (a) Slump Flow (Newman, Choo, 2003)   Figure 2.9 (b) J-Ring Test (Newmann, Choo, 203)

Depending on the application, the slump flow values can vary from 550 (for precast and flat applications) to 850 mm.

Bonen and Shah (2005), classified SCC as the powder type and VMA type based on the incorporation of the super plasticizers, low water: binder ratio and low aggregate: binder ratio. They found that the addition of super plasticizers by increasing the binder content showed low yield stress, increasing plastic viscosity, segregation resistance, and lesser inter-particle collision and blocking of the mix. Powder type SCC showed very similar properties to high performance concrete like high strength, low permeability, good freeze-thaw performance, high resistance to chloride – ion diffusion, high abrasion resistance because of its low w:c ratio , incorporation of mineral admixtures like silica fume , ground granulated blast furnace slag , limestone or fly ash. It was found that the fine to coarse aggregate ratio of SCC is higher than that of the ordinary concrete. The filling ability of the mix depends on the particle packing density.

Passing Ability: Passing of obstacles such as narrow sections of the formwork, closely spaced reinforcement etc. without blocking caused by interlocking of aggregate particles. Passing ability is normally measured by L-Box (Fig 2.10), or J-Ring.

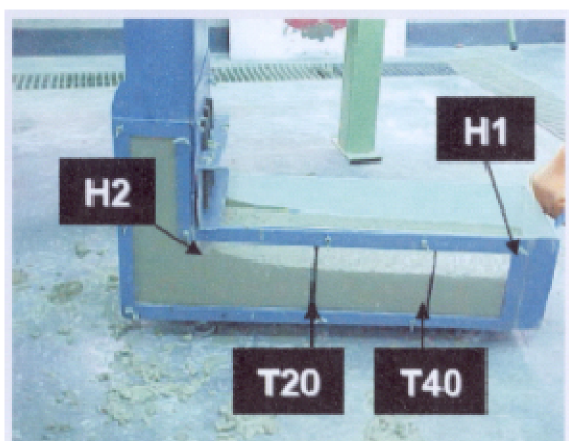


Figure 2.10: L-Box Test (Newman, Choo 2003)

Sonebi, Grunew, Walraven (2007) studied the influence of parameters like the dosage of HRWRA, water and ald the volume of coarse aggregates and their interactions on the filling and passing ability of SCC by various factorial and statistical design methods. It was found that by increasing the volume of fine aggregate and the paste enhanced the filling ability. The continuously graded cement and fillers reduced the inter-particle friction between the various solid particles (coarse aggregate, sand and powder materials), thereby affecting the filling ability. The use of fine materials like lime stone powder, slag cement, fly-ash, micro silica fume enhanced the grain size distribution and particle packing thereby increasing the cohesiveness. Factorial design methods were used to determine the influence of key mixture ingredients like water content, dosage of HRWRA, volume of coarse aggregates and their interactions on the relevant properties of SCC.

Various iso-curves and surface contours were plotted between slump flows, L blocking ratios, Oriment and V funnel flow times with T60 at different time intervals correlating the effects of CA, dosage of water and binders like HRWRA on SCC properties. The analysis of the derived models enabled the identification of major trends that can reduce cost, time and effort associated with the selection of trial batching for SCC.

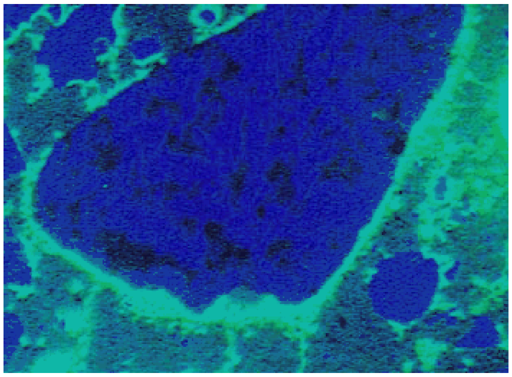
The models (statistical) established using a statistical design approach provided an effective means to evaluate potential mixture proportions by determining the influence of key parameters on the desired fresh properties necessary to obtain a good SCC. The L box test, which was influenced by the dosage of water, HRWRA and the volume of coarse aggregate, was most suitable to evaluate passing ability of SCC. The derived results at 60 min showed that the increase in the volume of the coarse aggregate significantly increased the V-funnel and Orient flow times whereas the increase in the dosage of water and HRWRA reduced the flow times.

Resistance to Segregation: Maintaining of homogeneity throughout mixing, during transportation, and casting. The dynamic stability refers to the resistance to segregation during placement. The static stability refers to the resistance to bleeding, segregation, and surface settlement after casting. It can be the most difficult property to quantify. It is normally checked visually, although there have been a number of attempts to quantify the segregation resistance. Chabib, Nehdi (2006) investigated the risk of segregation of the fresh SCC mixture in terms of grading of coarse aggregate, addition of HRWRA, incorporation of micro fillers and supplementary cementatious material. The main purpose of their research was to focus on the effect of the cementitious material content , w/cm , coarse –aggregate to total aggregate ratio (CA/TA) , and the dosage of HRWRA , VMA on the segregation potential of the SCC mix. To account for the effect of cementitious material content it was interpreted that while increasing the cementitious material (at constant 0.45 w/cm), the segregation tendency of the coarse aggregate increases in both the static and dynamic test whereas increasing the cementitious material (at constant 0.40 w/cm) slightly reduced the segregation tendency. This hindered the beneficial effect of increasing the volume of mortar compound by increasing the cementitious material content resulting in increasing the segregation of coarse aggregate particles in the mixture.In order to quantify the effect of HRWRA and VMA dosage, the authors found that for constant w/cm and VMA content the ability of the SCC mixtures to resist segregation linearly decreased with increasing the HRWRA dosage. Lastly, the authors recommended the CA/TA content to be limited to 0.5 in order to reduce the inter-particle friction between the coarse aggregate particles, thus enhancing the flow ability of the SCC. To conclude the authors delineated the relationship between the various effects of the basic mixture ingredients on the segregation resistance of SCC. They correlated the coefficient of variation (up to 10%) and the penetration depth of the coarse aggregate under both the static and dynamic segregation of SCC. The ability of SCC mixtures to resist segregation significantly decreases with the increasing w/cm and the dosage of

HRWRA. The VMA also found to be an effective tool for reducing the segregation resistance of SCC. However, the total content of cementitious materials had a mixed effect on the segregation mainly in the dynamic segregation; increasing the cementitious materials resulted in the increase in the segregation for SCC at high w/cm ratio, whereas in contrast with low w/cm ratio with increasing content of the cementitious material enhanced the ability of SCC mixture to resist segregation.

2.3.3 Hardened SCC

Composition of materials for SCC differs from the composition of materials for CVC. SCC is usually sandier, and the binder (cement plus filler) content is higher. Properly proportioned and executed SCC is generally more compact and less variable than equivalent CVC. A combined worldwide experience proved that that there is only a small difference between SCC and CVC hardened concrete properties (Newmann , Choo, 2003) of the same water-to-binder (water-to-cement) ratio, although some of those properties (e.g. compressive strength, bond to reinforcement, permeability, drying shrinkage, etc.) are enhanced in SCC. According to Bonen, Shah (2005), the modulus of elasticity of the SCC is smaller than the ordinary concrete as the elastic modulus of the aggregate is higher than the paste and the SCC has a relatively less amount of the coarse aggregate than the paste. Furthermore the shrinkage of SCC is more than the ordinary



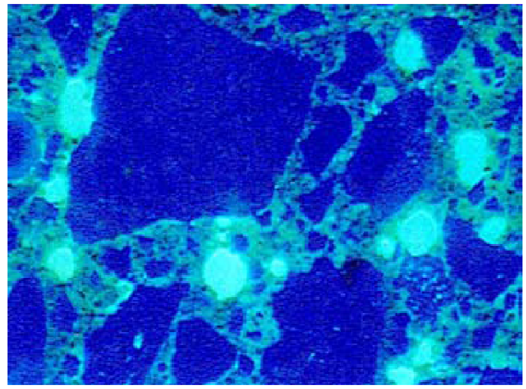


Figure 2.11(a): Microstructure of CVCv Figure 2.11(b): Microstructure of SCC (Newmann, Choo, 2003)

concrete because of the more paste volume as compared to normal concrete. The creep is higher than the normal concrete as the aggregate content is less than the paste in the SCC. Improvements in compressive strength and some durability characteristics of SCC (i.e. oxygen and water permeability) are related to the reduced porosity of the interfacial transition zone (between cement paste and aggregates) of SCC and general improvement of the microstructure of SCC compared with CVC concrete.

It was Assad, Khayat (2006) who evaluated the influence of the type and concentration of the viscosity enhancing admixtures (VEA) on the formwork pressure of SCC mixes. A liquid polysaccharide, powder polysaccharide and cellulose based VEA were used in the study. Both the naphthalene and polysaccharide based high range water reducing admixtures (HRWRA's) were employed. The thixotropy of the concrete was evaluated and correlated to the initial lateral pressure and its variation with time.

Tests results showed that the type, combination and the dosage rate of the VEA-HRWRA had a marked effect on the thixotropy and the formwork pressure. Secondly, irrespective of the VEA type and the combination with HRWRA , results indicated that the incorporation of the VEA type at a relatively low concentrations resulted in a lower formwork pressure compared with to the reference mixtures made without any VEA and those containing higher or medium concentrations of VEA. In addition the SCC made with a low VEA concentration necessitated lower HRWRA demand to secure the targeted slump flow. Finally, a good relationship was established between the formwork pressure and thixotropy for SCC mixtures containing the low concentrations of VEA with mixtures of greater degree of thixotropy exerting lateral pressure on the formwork. Also the effect of the water and cementitious material ratio (w/cm) and the type of the high range water reducing admixtures on the SCC mixes was evaluated to account for the development of formwork pressure. They assessed the effect of w/cm and the HRWRA type on the variations in the lateral pressure of the self-consolidating concrete. The variations in the lateral pressure were related to the thixotropy of the concrete. The rate of pressure drop and increase in thixotropy with time, however were greater in mixtures made with a higher w/cm. This was attributed to the lower HRWRA demand that lead to a sharper fluidity loss with time. Compared with the naphthalene and melamine based HRWRA , the use of poly-carboxylate based HRWRA in SCC resulted in a lower rate of pressure drop with time. This was reflected by the greater fluidity retention of the mixtures containing the poly-carboxylate based HRWRA. The incorporation of a water reducing agent in the mixtures made with poly-naphthalene sulfonate based HRWRA had increased the lateral pressure development of the plastic concrete over time. Thus, the results were of high interest mainly to the concrete technologists and contractors using the highly flow-able concrete.

Manufacturers and Pre-casters:

Concrete product manufacturers value the SCC technology more than most. There are a number of reasons for this: the benefits of SCC are evaluated and recognized faster and more accurately at a precast factory than on construction site; pre-casters are not restricted by the selection of the type of concrete and technology; precast production is a repeated process much more suitable to the implementation of automation. Though there are advantages of producing high quality SCC pre-stress members at reduced labour, their concerns about its effects on the hardened properties had limited its widespread applications. Schindler, Barnes, Roberts, Rodriguez (2007) evaluated SCC mixtures for use in pre-stress members, effect of mixture proportions on fresh properties, strength and modulus of elasticity at pre-stress transfer and the shrinkage behavior of concrete in full scale members. They found that the sand to total aggregate ratio had practically no effect on the value of characteristic compressive strength (fci) of the SCC mixtures. It was found that the higher compressive strength fci values were obtained by decreasing the water to cementitious material ratio.

2.4 ISSUE OF BOND IN PRE-STRESSED CONCRETE:

Researchers found it difficult to model a mathematical representation of the bond problem over the decades.

An assurance criterion for Flexural Bond in Pre-tensioned Hollow Core Units. Anderson and Anderson (1976).

The main purpose of the test was to investigate the flexural bond performance of the precast pre-stressed products. They tested 36 factory – produced hollow core units of spans ranging from 8 to 20 ft . In order to understand the strand slip in the members , they explained the concept of "free end slip". They found that the amount of "draw in" in the members after sawing is referred to as the "free end slip". They found that it is proportional to the transfer length, and it is an indication of the quality of the transfer bond development in a pre-tensioned member. They claimed excessive transfer length could lead to premature overlap between the transfer region and the flexural bond wave. Thus , to conclude the correlation between the members ability to achieve the transfer bond and its flexural bond strength , the free end slip serve as a predictor of the general bond failure. The ultimate strength of the member is directly related to the free end slip as this indicates the failure due to the premature loss of bond. The other factors that resulted in the failure are the amount of slippage , nominal diameter of the strands and the development length. They supported that the tests conducted at PCA, had shown that low concrete strengths like 2000 psi was not a primary factor in the bond performance. The transfer and flexural bond characteristics of strands proved that sudden transfer of pre-stress does not cause poor bond. They tested eight specimens with oil coating around it, does not exhibit poor transfer or flexural bond performance. To conclude, they found that one factor which displayed the free high end slip and the subsequently early bond failure was the poor consolidation around the offending strands. This was due to the partial collapse of the nearest void, by rock pockets in the vicinity of the strands, large pores left by "bleeding" between the layers in the products cast in stages.

Nature of Bond Pre-tensioned Concrete

Jack.R.Janney (1954)

The aim of the research was to investigate the methods and function of bond in the pre-stressed members. The main purpose of the study was to investigate a series of questions like the extent of influence of wire diameters on the transfer of pre-tension from the steel to the concrete, the extent of the effect of surface conditions of the wires and strands on the pre-stress transfer bond properties, effect of the concrete strength on the transfer of stress from the pre-tensioned steel to the concrete and the importance of bond in resisting the external bending moments and whether the properties of the flexural bond are influenced by the pre-stress transfer bonding existing near the beam ends. He found that the adhesion i.e., the no slip condition can only take place if the reduction in the tensile strain occurring in the steel at any point after the pre-stress release equals the compressive strain in the concrete at the same point. At the ends, there is more reduction in the steel strain than that of the concrete strain, with the difference more predominant at the ends. He found that at the ends the reduction in the steel strain was maximum and the corresponding concrete strain is zero, indicated that there must have been an end slip and the factor of adhesion can be discounted as the major factor of the pre-stress bond.

Thus he claimed that the fiction between the concrete and the strand is the main factor responsible for the transfer of stress to the concrete. The coefficient of friction will vary with the surface characteristics of the wire and the paste. He found that the length of the embedment necessary to transmit the stress fully to the concrete was moderately greater as the wire diameter increases. To avoid the variation in retained tension , it was necessary to tension the wires higher or to increase the concrete cross section as the wire diameter increased. The other variable in the study was the concrete strength. As he found that concrete quality influences the ability of the concrete to sustain the radial pressure resulted from the increase in wire diameter, however frictional phenomena being the major contributor in the bond quality, the paste quality had hardly any effect in the bond. In order to find the effect of surface condition on the bond, he tested both the rusted and clean wire in the beams. He found from the beam tests that the full effect of the rusting was not reflected in the bond stresses developed at the release of the prestress transfer and the flexural bond properties of the rusted wire were greatly superior to clean wires due to the increase in the coefficient of friction. In order to account for the ultimate flexural bond stresses, rusted and clean wires were used in the beams and tested for the ultimate loads and the cracking loads. The comparison showed that the ultimate load of each beam reinforced wit the clean wires were little above the cracking load regardless of the amount of pretension in the wires. These beams the ultimate load was dependent on the cracking load irrespective of the amount of prestress in it. On the contrary the beams with the rusted wire failed by steel fracture and the ultimate load was dependent on the strength of the wire and not the degree of pre-stress. His research concluded that there were variation in the anchorage length and shape of the stress transfer distribution for wires of different diameters and different surface conditions ranging from clean to rusted. There were also variations, when the pre-tensioned steel was released to concretes of different strengths. He concluded by an elastic analysis of deformation that the pre-stress transfer bond was largely as a result of friction between the concrete and the steel. For the flexural bond stresses , it was found that the high bond stresses develop only after the cracking had occurred and the beam fails due to the loss of bond, he concluded that the pre-stress beam will carry greater ultimate load than an unpre-stressed beam reinforced with the same steel.

Use of large tendons in the Pre-Tensioned Concrete.

Neils Thorsen (1954)

The research showed the importance of bond properties in the prestressed members both at the ends and at the mid span. His research showed the significance of the transfer length and ultimate length needed to develop the fully effective pre-stress force. The graphs plotted against the steel stresses and the distance from the free end with the increasing slopes (bond stress) showed the importance of the transfer zone and the bond properties associated along with it. He found that at the time of transfer of pre-stress, the tendon swells due to the Poisson's ratio for the steel and causes compression on the interface between the steel and the concrete. He concluded that the bond stresses due to compression was more than the stresses due to tension. Lack of data and experimental test results inhibited him to correlate between the transfer length and the end slip. Though the assumptions were based on two simple concepts of friction and elastic bond. He also found that the bond properties do not depend on the steel type alone, but also on the initial stress level in the steel and quality of the concrete. In order to account for the secondary stresses generated at the end zones due to swelling of the tendons during release, the concrete around the tendon takes the form of the thick walled cylinder matching the shape of the tendon. The tensile stresses were maximum at the very end of a member and decreases over the transmission length. Thus to minimize these tensile stresses he called for further research on the spacing and nature of distribution of the strands.

Effect of Age of Concrete on Bond Resistance.

Peattie and Pope (1956).

They performed a series of tests which made the effect of age on the factors controlling the bond resistance in the adhesive and the frictional stages. The factors, which determined the bond resistance both in the adhesive and frictional stages, attain their steady values rapidly. The growth was more rapid in the frictional than the adhesive stages. The pull out test performed in order to quantify the effect of bond proved that they attained their bond strengths more rapidly than the torsion specimens as the mass the concrete and the length of the embedment was more in the pull out tests. The growth of the bond resistance was more rapid than that of the concrete strength. He concluded that the bond resistance was caused mainly by the shrinkage of the concrete closely adjacent to the steel. Due to the exothermic effect in the hardening concrete and the thermal insulation provided by this material the development of the bond resistance was relatively more rapid.

Flexural Bond Tests of pre-tensioned pre-stressed beams.

Hanson and Kaar (1965)

The major variables in the study were the embedment length and diameter of the strands with respect to their influence on the bond performance of the pre-tensioned pre-stressed beams. They correlated the variable involved in the flexural bond stresses. They found that when a strand was stressed to fracture at the ultimate load of the beam there must be a critical embedment length to be provided in order to avoid the strand slip. If the beam contains a high percentage of steel (or low concrete strength) then the flexural failure occurs by the crushing of the concrete though the steel was stressed below its ultimate strength. Thus, the bond slip did not occurred even if the embedment length provided was less than the critical length needed for the size of the strand used to develop the ultimate strand strength. Their research kicked in the correlating effect of concrete strength, percentage of steel and embedment length. During the beam tests they found that the beams with higher embedment length failed in flexure by crushing of the concrete after the yielding of the steel before the general bond slip occurred. With the decrease in the embedment length the failure was evident at the lower moments due to the slippage of the strands. The failure by the slippage of the strands occurred in the two stages firstly the general slip of the strands along its whole embedment length and secondly the destruction of the mechanical interlocking effect between the strand surface and the surrounding concrete. Thus, they suggested that for the static loading with a shorter embedment lengths an increase in the loading was possible, in contrast with the dynamic loading the load carrying capacity was reduced significantly after the bond slip. They claimed that the high percentage of steel reduces the bond failure as the steel stresses are comparatively more than a beam with low percentage of steel. They found that with reduction in the concrete strength there was a decrease in the steel stresses in flexure resulting in a lower bond stresses over the embedment length. By beam testing they found that the reduction in the average bond stresses due to the drop in the steel stresses was greater than the reduction in the bond strength due to the drop in the concrete strength then a failure due to the general bond slip was less likely in the beam with reduced concrete strength (low concrete strength failed in flexure). In short an increase in the reinforcement percentage or a reduction in the concrete strength reduced the possibility of the bond slip, as the steel stresses at flexural failure and the bond stresses are reduced. The rusting of the strands raised the moment at the general bond slip and the ultimate moment of resistance relative to the beams with clean smooth strand. Lastly, they concluded that the end anchors did not become effective until the slip had occurred along the entire embedment length and they hardly restrain the onset of general slip.

Influence of concrete strength on strand transfer length.

Kaar, Lafraugh and Mass (1963).

The research reported an investigation of the influence of the concrete strength on the stress transfer length of the seven wire strand at the time of the transfer of pre-stress. They found that the concrete strength had very little effect on the transfer length for the strands up to ½ in diameter. They tested four different strengths of the concrete with four different strand sizes. The specimens were concentric and rectangular. They found that in the case of higher diameter strands like the 0.6 in , the transfer length reduced with the increase in the concrete strength at the cut end, but did not show a clear cut relationship for the dead end. It was evident that the average transfer length at the cut ends of the specimens were approximately 20 % greater than at the dead ends for strands of 0.5 in diameter and lower and 30% greater than the 0.6 in diameter. The amount of increase in the transfer length with time does not appear to be related to the concrete strength at the time of transfer. The concrete strengths in the specimens showing the greater increase in the transfer length ranged from 2500 psi to 5000 psi for different diameter of the strands. They noted that the length of the specimen over which the prestress is transferred measured from the point at which the concrete strains first rise above zero was almost the same for all the specimens prestressed with 0.6 in diameter strand. At the dead end of the specimens the concrete strain commenced to increase immediately, indicating no slip occurred in the end regions where the pre-stress was transferred to the concrete gradually. The local slip of the strands at the cut ends of the specimens were apparently due to the sudden transfer of the pre-stress associated with the flame cutting of the strands. It was also observed that the local spalling occurred around each strand on the end face at the cut ends of the specimen prestressed with 0.6 in diameter. No such spalling was observed where the prestress was transferred gradually. They claimed that the strands upto 0.5 in diameter demonstrated the more rapid build up of prestress close to the ends of the specimens made from the higher strength concretes even though the distance to achieve the full prestress did not vary systematically with the concrete strength. Finally , they concluded that the average increase in the transfer length over a period of one year following prestress transfer was 6% of all the sizes of the strands. The maximum increase in the transfer length was 19%. The increase in the transfer length with time was apparently independent of the concrete strength at the time of transfer.

Effect of strand Blanketing on Performance of Pre-tensioned Girders.

Paul Kaar and Donald D.Magura (1965).

Two methods were used in the study for the limitation of compressive and tensile concrete stresses near the ends of the pre-tensioned members. Some of the pre-tensioned strands may be "harped" that means that they can be deflected upwards near the ends of the members (expensive process) or the other one was prevention of the bond to the concrete at the ends either by plastic tubing or "blanketing". The purpose of their investigation was to explore the possible effects of blanketing on the flexural behavior at the service loads and on the ultimate flexural, bond and the shear strength of the pre-tensioned pre-stressed girders. Under the service loads, the stresses in the pre-stressed reinforcement normally remains near the pre-stress level and the flexural bond stresses are negligible. For loads to the ultimate flexural strength, the stresses in the pre-stressed reinforcement must eventually increase substantially beyond the pre-stress level. If inadequate embedment (development) length was provided the ultimate strength was governed by bond failure rather than by flexure. They also claimed that the mechanical interlocking provided substantial force even after bond slip. Tests were conducted to study the influence of the flexural and flexure/shear cracking on the development length. Only three girders were tested and the strands used were 3/8 in diameter. Beams were fatigue loaded to 5 million cycles, then they are statically loaded to ultimate. Of the three beams, one was fully bonded, one was partially bonded and one was fully de-bonded. In the tests to ultimate there was no significant difference in the behavior of the first two beams. In the third beam however the ultimate strength was reduced by about 15%. The failure on the third beam did not occur at the maximum moment (where the first two failed) but at a point 7.5 ft from the maximum corresponding to the de-bonded strand.

Influence of surface roughness of pre-stressing strands on bond performance..

Hanson (1969).

Their investigation evaluated the effect of the surface roughness on the bond strength of the strands in the pre-tensioned beams with seven wire strands. The comparison was made for the bond strength in the "as received" condition with that of the partially rusted, rusted and deformed (dimpled) strand. They found that for the members which needed high bonding capacity both the transfer bond and flexural bond for short cantilevers, short flexural members railroad ties, truss members, footing beams etc, surface roughness of the prestressing strands provided much improvement. The release of pre-stress by cutting strands with a torch or grinding wheel increases the transfer length. They recommended further studies to determine if there were a problem of shock due to the energy release or with the stress increase in the strands during the release. They found that the flexural loading produced high steel stresses in the transfer region of the members reinforced with the smooth strand led to the failure by bond slip at bending moments substantially below the ultimate flexural stress. The flexural strength at the beam ends was increased by shortening the beam ends. These showed that both the surface rust and surface deformations effectively increased the flexural strength by improving the bond. Earlier , it was concluded that the bond transfer length could be reduced by using a small strand diameter or by providing a gentle and gradual release of the pre-stress.

Bond Strength as a function of strand tension and Cement Paste Content for Lightweight Aggregate concrete.

Tulin and M.Al-Chalabi (1969).

They studied the effect of strand tension and cement content as a function of the bond strength. Several concrete mixes using the conventional sand were used in the investigation. They assumed the bond strength as a function various parameters like characteristics compressive strength of concrete, strand tension, reference strand tension, absolute volume of cement paste and absolute volume of sand and lightweight coarse aggregate. They developed empirical and analytical relationships in reference to a surface which could be interrelated as a tension ratio and cement paste ratio. Results showed that a decrease in the pre-stressing force was accompanied by a slight increase in the bond strength while an increase in the cement content produced a slight decrease in the bond strength over the range of variation parameters considered.

Bonding Properties of 0.5 in Diameter Strands.

Edwards and Picard (1972).

The purpose of the investigation was to determine the bonding properties of 0.5 in diameter strands and verified the theoretical results obtained after the experimental tests.

They studied the cracking mechanisms of the concrete member, a theory that was developed to predict the spacing and width of the cracks using the bonding properties of the reinforcements defined by the "bond slip" curve. From the bond stress slip relationships they found that when the bond stresses were reached or close to its maximum value the longitudinal crack appeared. When the strands which were embedded in the concrete elongated, bearing action occurred between the outside wires and the concrete matrix filling the helical groves of the strand. This bearing pressure was enhanced if the torsional movement of the strand was prevented. In the tests reported , the concrete specimen was free to rotate with the strand thereby the additional contribution to the bond was absent and the measured bond was lower than that obtained in the tension member tests. Stocker and Sozen proved that the torsional stiffness had a little effect on the bond. They concluded that the bond relationship for the strand was almost linear up to the critical slip. For further increase in the slip, the bond stresses remained constant and equal to its maximum value provided no longitudinal crack appeared in the specimens. The average value of the maximum bond strength decreased when the concrete cover increased due to the settlement of the concrete under the strand which was held in a horizontal position during the casting. The crack width depended mainly on the steel stresses at the crack and on the maximum bond strength of the reinforcement. The crack width increased with the steel stresses, strand diameter and decreased with the increasing bond strength. The predicted average crack width and predicted experimental scatter compared in accordance with the experimental results.

Bond fatigue tests of beams simulating pre-tensioned concrete crossties.

Paul H.Kaar and Norman Hansom. (1975).

The research used three types of strand conditions smooth , rusted and sandblasted. Both the gentle release and sudden release were used for detensioning.

The average values of the transfer length showed that the smooth strand had a longer transfer length than the rusted and abraded ones. Also the sudden release of the prestress resulted in the longer transfer lengths. It was found that the cross-sectional properties greatly influenced the measured transfer length. The number of strands greatly influenced the transfer length. The larger the number of strands for a cross section, the less scatter there is in the data and more accurately it could be predicted. They found that the size of the cross section influenced the transfer length itself. To conclude the larger cross sectional area, shorter the transfer length. Their research concluded that the smooth strand did not performed well and resulted in the bond failure even before the first cycle was reached. Thus the surface condition of the strands effectively reduced the required transfer length. For the bond failure to occur before cracking load was reached, the point of load or the point of maximum moment must be placed in or very near to the transfer zone**.**

Development of Pre-stressing Strands.

Paul Zia and Talat Mostafa (1977).

They performed extensive research in the development of bond and equations for the transfer length of pre-stressing strands. Several investigators had formulated theories for the transfer length based on different concepts of bond between steel and concrete like wedging action, friction, friction plus shrinkage or certain assumed bond slip relationships. Their validities were questionable as they are based on elastic concept and highly localized concrete stresses are generated within the transfer zone.

The various factors affecting the transfer lengths are type of steel (wire, strand), steel size (diameter), surface conditions of the steel (clean , oiled, rusted), concrete strength, type of loading (static , repeated and impact), type of release (gradual, sudden , flame cutting, sawing), confining reinforcement around steel (helix or stirrups), time dependent effect , consolidation and consistency of concrete around steel and amount of concrete coverage. As most of the parameters were not properly and uniformly quantified they were left for further research in the bond studies. It was concluded that the transfer lengths were longer for larger steel sizes, higher pre-stress level and lower concrete strengths. Sudden release of pre-stress by flame cutting or sawing increased the transfer length. Since the strands provided a certain amount of mechanical resistance in addition to friction their transfer length were shorter than the smooth wires. Under repeated loading applied outside the transfer zone , no significant effect on the transfer length were observed. If applied within the transfer zone repeated loading caused early bond failure if the crack was developed within or near the transfer zone. The use of reinforcement to resist the bursting stresses near the end of the pre-stressing steel reduced slightly the transfer length, although the effect was not significant. They found the transfer length were increased for small size wires, however, there were virtually no change in the transfer length with time.

Fatigue tests of pre-tensioned girders with blanketed and draped strands.

Kaar, Russell, Bruce Jr (1979)

They performed fatigue tests on full-sized AASTHO bridge girders and found that to control stresses in the end regions of the pre-tensioned members, straight strands having un-bonded blanketed lengths at the ends of the girders could be used effectively and economically as an alternative to draped strands.

For similar loading conditions they found that the behavior and strength were the same for the girders having either blanketed or draped strands. The fatigue life of the specimens designed for a maximum tensile stresses of 6(f'c)^0.5 under the full service load was significantly less than that of the specimens designed for zero tension and had blanketed strands designed for twice the development length only small slip of the strands occurred. This indicated adequate bond of the blanketed strands for about 3 million cycles of repetitive loading. Also the blanketing did not cause the fatigue of the strands. They found that the use of ties to confine the concrete in the stress transfer region of the blanketed strands in one specimen did not provide any substantial improvement in the behavior of the specimens. They recommended that the development length greater than ld without exceeding the allowable concrete stresses, would result in less length of the strands to be blanketed and more economical to manufacture. When the tension was allowed in the concrete under service load conditions design of the girders to prevent strand fatigue must be considered. Further , research was needed to determine the fatigue properties of the pre-stressing strands as well as the level of tension in the concrete at which pre-tensioned girders would be able to withstand traffic loading without the strand fatigue during their design service. They suggested further research to determine the distance till which the pre-stressing strands could be extended beyond the point where they were not needed.

Use of de-bonded strands in Pre-tensioned bridge members.

Daniel Horn and Kent Preston (1981).

They illustrated the methods of debonding. The test results of the pretensioned members with debonded tendons were summarized and a design procedure was established.

The research also investigated the effect of debonding on the strength and behavior of the prestressed concrete girders subjected to repetitive loading. They determined the length required to develop the strands in the debonded construction, whether the tension in the concrete under service load condition affects the development length, whether "wrapping" to confine the concrete in the stress transfer region of the debonded strands were beneficial. They concluded that debonded members designed for one development length and zero tension stress under the service load conditions exhibited a strength equal to girders with bonded draped strands. The de-bonded members designed for a maximum tensile stresses in the concrete of 6*(f'c)^0.5 and one development length failed in bond fatigue while those members designed for twice the development length exhibited equal strength when compared to draped strands. The use of "wrapping" to confine the concrete in the stress transfer region of the debonded strands did not provide any substantial improvement in the strength or serviceability of that specimen. They further recommended further research to determine the length to which the strands must be extended beyond the point where they were theoretically no longer needed. They concluded that out of the various techniques of debonding like greasing, chemical retarders, taping, split sheathing and solid sheathing; the solid sheathing gained popularity. This eliminated the long sections of the taping that the split sheathing required. The sheathing was pushed over the strands however the added time to perform the operation was less than that required to tape the split sheathing.

Bonding and Corrosion Protection Properties of two coatings for prestressing steels.

Fernand Ellyin and Rafik Matta (1982).

The aim of the research was to study the effect of corrosion resistance and bonding properties of the two types of coatings.

The cement slurry and the coal tar modfified epoxy resin were applied on the different types of prestressing strands. Twenty pretensioned and post tensioned beams with uncoated, cement slurried, resin coated strands were subjected to the attack of calcium chloride and sulphur dioxide environments. They concluded that the protective coating having an undesirable anti corrosive qualities might affect the bonding strength of the prestressing strands to the concrete. Such an effect must be taken into consideration by insuring a sufficient bond length for the coated strands. The cement slurry provided an excellent bonding mechanism to concrete for smooth, straight wires. It also provided a good protective layer for the steel. However, it must be ensured that there were no cracks in the applied coating by adding inert plasticizers. The concrete strengths affected the bonding strength of the embedded coated wires. The higher the strength , the better the bonding, in the case of smooth wires it was less appreciable. In the post-tensioning system , the pre-stressing steel of left without any kind of protection even for a shorter time in a moist corrosive environment, it will undergo substantial mechanical changes(strength loss and ductility), which will lead to undesirable results.

Pre-stress Transfer Bond of pre-tensioned strands in concrete.

R. Stanton Over and Tung Au (1981)

The main objective of the research was the diameter of the strands and the bond transfer length required for each strand diameter was studied. The tests were made on the specimens with the pre-tensioned strands of ¼ in , ¾ in and ½ in diameter and also on a specimen with single wire of 0.250 in for the purpose of correlating data. For any given distance from the free end, the average stress in the concrete was proportional to the stress in the pre-stressing strands, regardless of the range of stresses and pre-stressing loads. They compared the concrete surface stresses to the steel strand stresses at the same distance from the free ends. The concrete stresses increased linearly with respect to the interior steel stresses to a point as the pre-stress load was applied. Then the steel stresses were decreased at a faster rate compared to the increase in the concrete stresses. Also they noticed that the concrete stresses diminished as the steel stresses decreased. This proved the fact the seven wire strand, in contrast to the individual wires, developed additional stresses in the concrete after they had slipped. This is due to the mechanical bond resistance was an important characteristics of the strand performance in pre-stressed beams. To conclude they suggested that the transfer length required for the strands of larger diameter was greater than that of the smaller strands. They found considerable difference in the stress distribution and transfer length found by gages applied directly to the strands and concrete surfaces, and concluded that data obtained from strand gages were more accurate. The multiple wire strands required lesser transfer length than single wire strands of equal strength and stresses.

Pre-stress Transfer Length.

R.E Loov and R.Weerasekera (1990).

The study showed that the improved transfer length predictions could be made if the effect of concrete strengths, pre-stress, cover, spacing were taken onto consideration in the 50 times strand diameter equation. On the basis of Janney (1954) , who concluded that the concrete near the tendon would have radial compressive stresses exceeding the elastic limit so that inelastic deformation occurred, they performed an analytical study. They considered that the transfer length had three zones, a zone nearest the end with the radial cracks extended to the surface, an intermediate zone with the radially cracked concrete restrained by an annulus of un-cracked concrete and a third zone which is elastic and uncracked. The radial cracks were assmed to be sufficiently narrow so that some tension continued to transfer across them. They concluded from the numerical analysis that the prestressing steel expanded so much in the transfer region that the concrete developed radial cracks along most of the transfer length. The expected width of the cracks in the transfer zone fell within the "cohesive cracking" range which permitted the tension to continue to the transferred zone across them. They concluded that the analysis was sensitive to small changes in diameter, any irregularities or surface roughness producing an effective increase of diameter during slip may be subjected to importance. From the equilibrium analysis, they found that the transfer length could be predicted by considering simple transverse and axial equilibrium provided the average concrete tensile stresses and coeeficient of friction can be estimated. For each strand size and level of prestress there appeared to be an optimum cover which was most effective. However, the biggest drawbacks of his study were assuming the strain compatibility at the steel and concrete interface and assuming pre-stress as a smooth cylinder thereby disallowing any mechanical interlock.

B. W. Russell (1992)

According to Russell (1992), that the tangential stresses in the concrete surrounding the strand exceeds the tensile capacity of the material, causing the concrete to crack locally and creating a material discontinuity. Also the friction factor has been recognized as a major contributor to the transfer and development of the pre-stressing force, but there have been very few efforts to quantify the frictional bond stresses between the pre-stressing steel and concrete. Thirdly, the helical nature of the seven wires contributes to the bond stresses by the mechanical interlocking (Russell, 1992).

SCC has drawn attention in pre-stressed industry due to benefit in enhancing construction productivity. Despite the interest in SCC and rapid development of SCC technology, its widespread use is restrained somewhat by the material and structural performance concerns, including the issue of bond.

The concept behind the pre-stressing and reinforced concrete clearly relies on the ability to transfer tensile forces from the strands into the hardened concrete both during service as well as at ultimate. This forms the basis of the design consideration of the precast/pre-stressed members through the formulations of the development length to ensure the proper anchorage of the pre-stressing strand when relying only on the interaction between the strand and the surrounding concrete. These transfers of stresses from the strand to the concrete in simple terms are done by three basic mechanisms of materials. They are:

- Adhesion.
- Hoyer's Effect
- Mechanical Interlocking.

2.4.1 Adhesion: It is the tendency of the certain dissimilar molecules to cling together due to attractive forces. This happens due to the differences in the surface of the strands and wires cause the mechanism of transmission of the pre-stressing force from the reinforcement to concrete to vary. According to Russell (1992), the glue between the concrete and the steel is rigid-brittle. The failure of the glue is brittle. This means that it prevents displacement of the strand relative to the concrete until a certain critical stress is reached. At this critical stress, the glue fails and the resistance reduces to zero. Due to this rigid-brittle behavior, this hardly contributes little or nothing to either pre-stress transfer bond or the bond developed to resist additional strand tension from the applied loads.

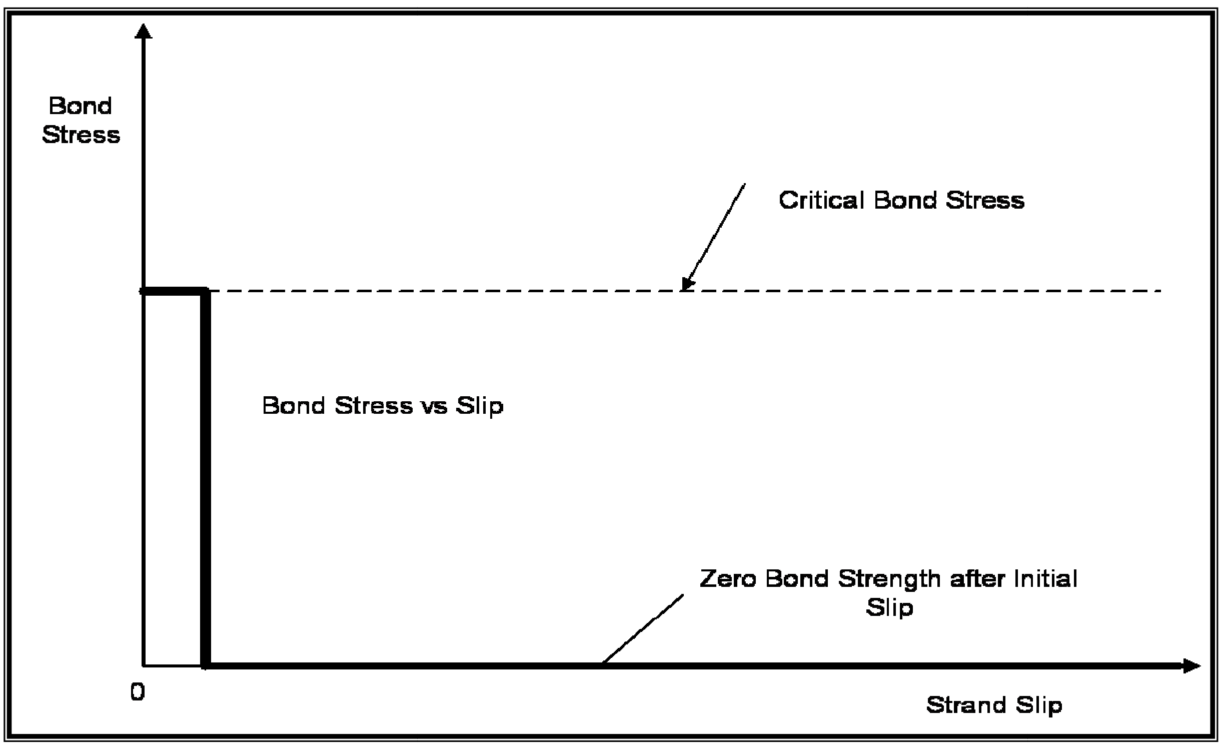


Figure 2.12: Adhesion Effect (Russell, 1992)

2.4.2 Hoyer's Effect: In 1939, E. Hoyer investigated the mechanism of stress transfer from the pre-tensioned steel to the concrete. He found out that lateral pressure between the pre-tensioned steel and the encasing concrete mainly influences the bond behavior within the transfer length.

The steel is elongated due to the pre-tensioning which results in a lateral contraction of the steel. Upon release, the steel tries to return to its unstressed dimensions but the hardened concrete inhibits the expansion of the steel. High lateral pressure arises between steel and concrete while stress is transferred. The radial pressure dominates the transfer of the pre-tensioning force and the bond strength of the steel, especially for smooth strands. This effect is known as the “Hoyer Effect” or as “Poisson’s ratio effect”, as shown below.

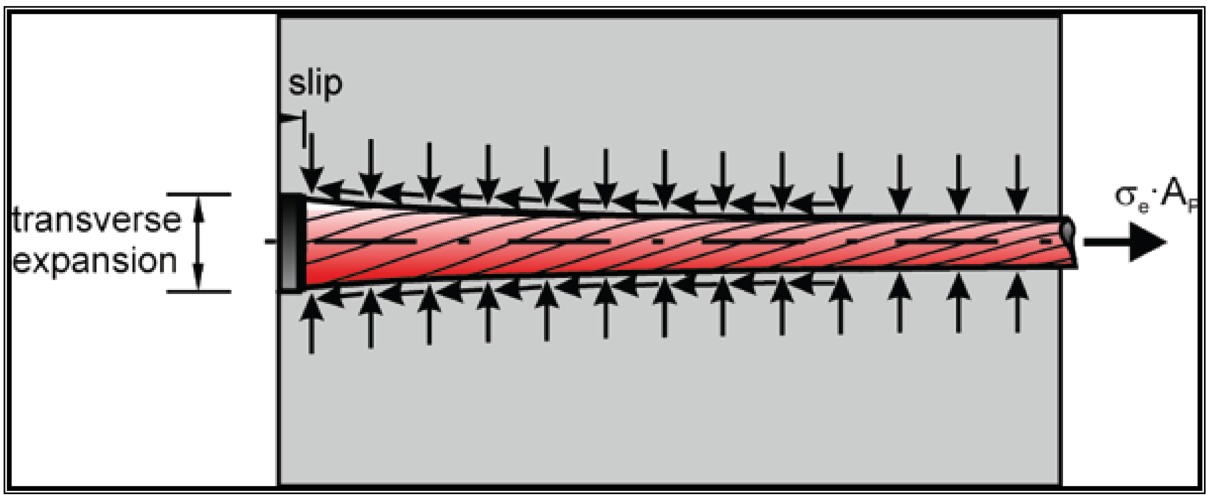


Figure 2.13: Schematic of Hoyer Effect (Russell, 1992)

According to Hegger, Will, Butte (2007) the bond anchorage of the strand is mainly a function of three main parts:

- Base Value.
- A stress – dependent part.
- A slip-dependent part.

The base value of the bond is a combination of the adhesion and base friction. The stress-dependent part results from the additional friction due to the lateral pressure between steel and concrete as explained by Hoyer Effect. Due to slip, a strand moves through the concrete and follows the pre-shaped channel. The slightly irregular geometry of the strands (e.g the variation of the pitch) restrains the strand. Hereby, the bond stresses increases (slip dependent part).

At the top of the figure, a concrete specimen with a strand after the release of the pre-tensioning

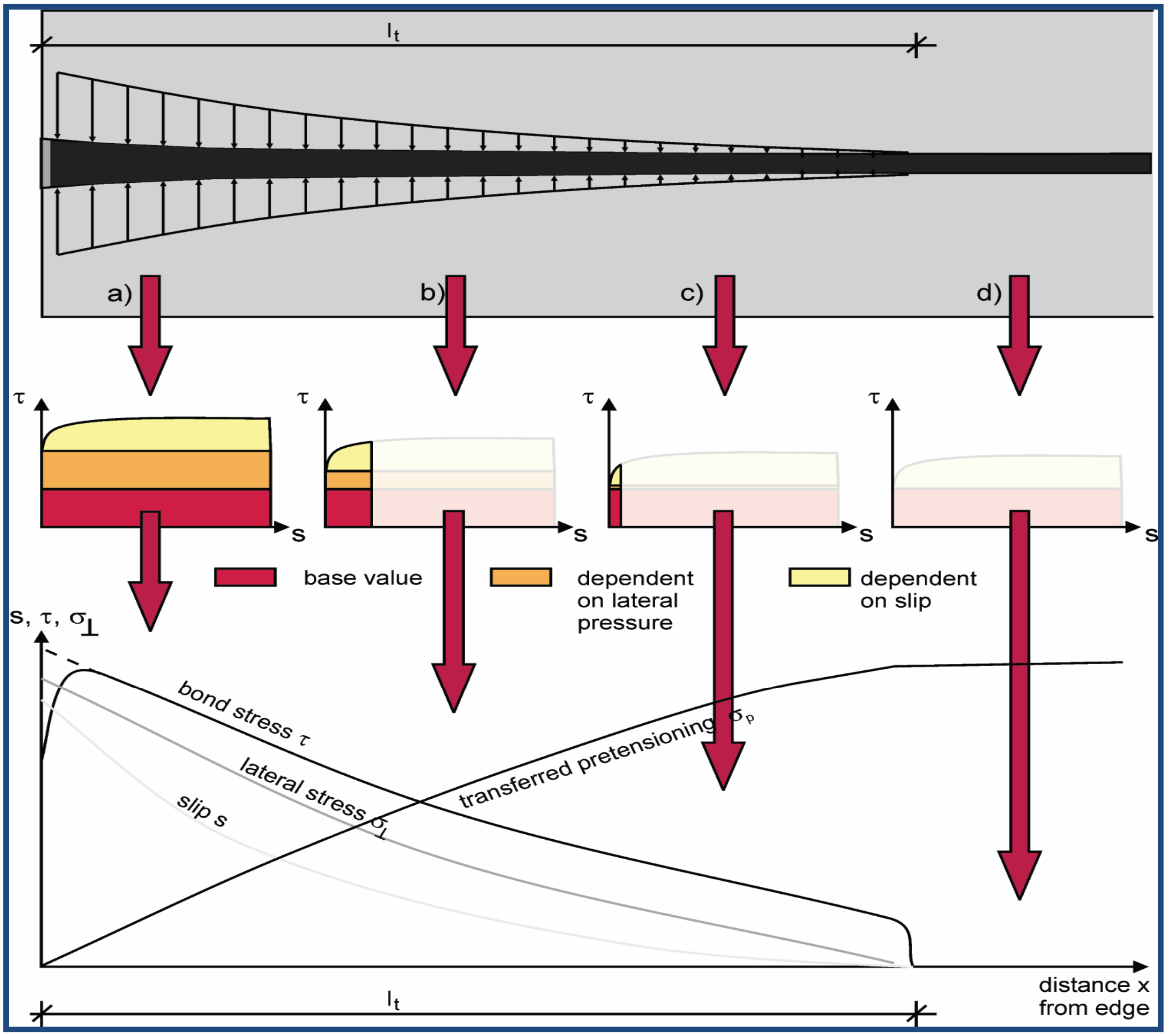


Figure 2.14: Correlation of Bond and Transfer Length (Josef Hegger,Nobert Will,Sebastian Bulte, 2007)

is shown including the expansion of the steel, the resulting lateral pressure and the end slip. The schematic bond slip relationships and the bond strength related to the existent bond slip are displayed for different areas along the transfer length. At the bottom, the development of the bond stress, lateral stress, slip and the transferred stresses are shown along the transfer length.It is interesting to note that the bond stresses are not constant along the transfer length. The slip as well as the lateral stresses arises from the difference between the steel and the concrete strain when the pre-tensioning is released.

As long as the steel and concrete are not in equilibrium, the steel continues to deform relative to the concrete, which results in slip and lateral pressure. The resulting bond stresses transfers the pre-tensioning to the concrete. Thus, close to the end of the member, almost the full pre-strain is re-covered (Figure: 2.14 area a). Furthermore, a high lateral pressure occurs between the steel and the concrete to establish equilibrium between the steel and the concrete. Thus, besides the base of the bond, the stress dependent part as well as the slip dependent part is fully activated. Along the transfer length, the pre-stress in the concrete increases and the stresses to be transmitted decreases as shown in the diagram (Figure: 2.14, area b). Therefore, the slip and the radial pressure between steel and concrete are reduced compared to the beginning of the transfer length. This reduction leads to a reduced bond stress. The figure explained that the slips as well as the stress dependent part are less pronounced. Near the end of the transfer length (Figure:2.14 area c) only little additional stresses has to be transferred from the steel to the concrete. In this area, the slip and the concrete pressure are very low. Thus, only little more than the base value of the bond strength is activated. Outside, the transfer length the steel and the concrete are in equilibrium without the stress transfer. Neither the bond stress nor the lateral pressure nor slip occurs outside the transfer length due to the pre-stressing of the concrete.

2.4.3 Mechanical Interlocking: According to Russell (1992), the concrete forms an envelope or sleeve surrounding the seven wire strand. The hardened concrete mimics the shape of the seven wire strand. The concrete surrounds the strand filling the narrow crevices between the individual wires. These crevices are called the interstices of the strand. If the strand attempts to pull through the concrete without twisting, the movement is resisted by the concrete ridges acting on the outside of the wires of the strand. This resistance is called mechanical interlocking. In a seven wire strand, the six outside wires are wound around one single center wire in a helical pattern. The helical windings provide the “humps” necessary to develop mechanical interlocking in pre-tensioned strand. When the tension is applied to the strand, movement of the strand relative to the concrete is resisted to the interlocking of the outside wires reacting against matching deformations in the concrete. This is illustrated in the diagram below (Russell, 1992):

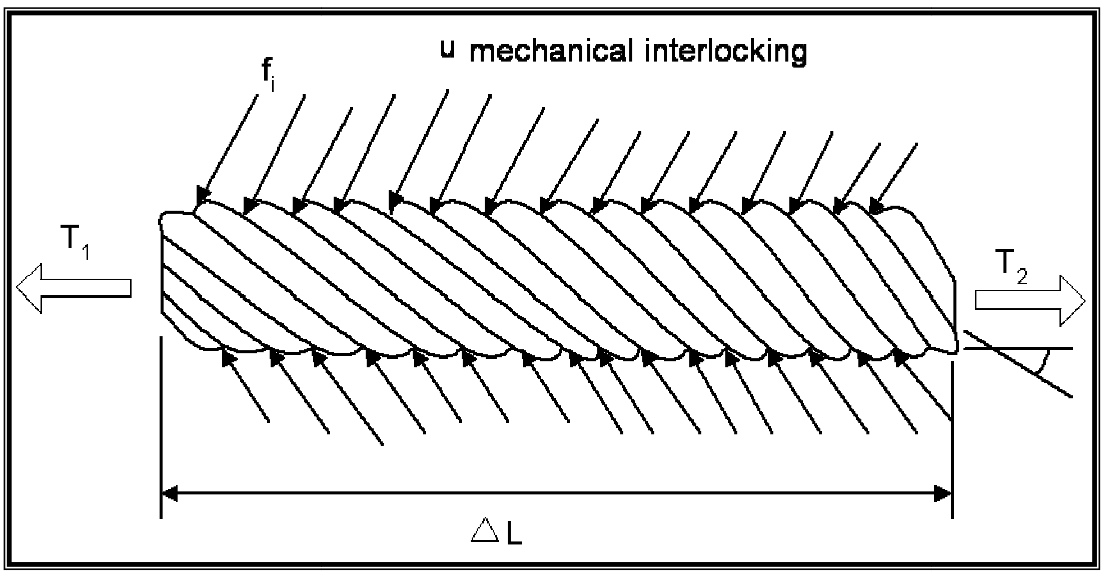


Figure: 2.15: $T_2 = T_1 + P_{ps} * \int_{\Delta L} U_{min}\, dl$ (Russell, 1992)

Where, $U_{min} \rightarrow f\,(f_i, \sin^2\theta, \mu)$

Pps = Strand Perimeter

μ = Friction Coefficient

Significance of interlocking: The mechanical interlocking is the largest contributor to the flexural bond mostly in the cracked regions. It was found that as the crack forms, the strands slip must occur for some small finite distance on either side of the crack to preserve the compatibility of the strand. When the slip occurs, the mechanical interlocking is activated by the reaction of the outside wires interlocking with the concrete envelope. Bond stresses from the mechanical interlocking can be very large in the immediate vicinity of cracking. The flexural bond stresses results from the changes in stresses in the steel. Thus, at the crack locations rapid increases in the steel stresses demonstrates the high bond stresses.

2.4.4 Structural Significance of Pre-stress Bond

Transfer Bond Stresses:

The distribution of the bond stresses τ at the transfer of the pre-stressing force is mainly due to the combination of the mechanical interlocking and Hoyer's effect. The transfer bond mainly comes from the Hoyer's effect as the twist restraint is the origin of formation of the mechanical interlocking to develop fully effective. The increase in the bond from the mechanical interlocking develops as the twist restrained is generated from the Hoyer's Effect. The approximate distribution of the bond stresses at transfer due to these mechanisms, where bond stress becomes zero, the stress in the strand becomes equal to the stress due to the pre-stressing. The length associated with this is termed as the "bond length". It will depend on the quality of the bond and on the transverse pressure determined by the member geometry and transverse reinforcement. The pre-stressing force is introduced into the member until the concrete stresses exhibit a linear distribution over the section. The length needed for achieving this is referred to as the "transfer length".

At the face of the structural member, the steel and the concrete stresses are zero. The shear or the bond stresses between the strand and the concrete increases rapidly until it reaches its maximum value, beyond which it decreases as per a parabolic curve.

Due to the bond effect, the compressive stresses radiate from the wire into the concrete causing warping at the member ends. As a consequence of this, a zone of compressive stresses develops acting radially towards the strand. This further enhances the “Hoyer Effect”.

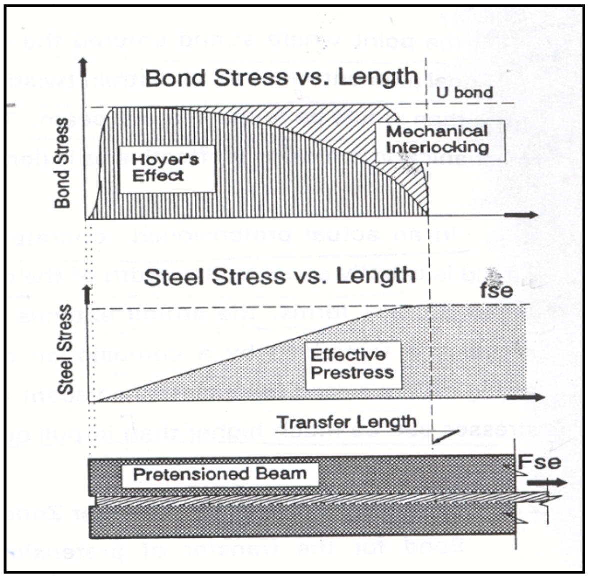


Figure 2.16: Bond Mechanics in Transfer Zone (Russell, 1992)

Moreover, this deformation also introduces tensile stresses which call for the need of the transverse reinforcement.

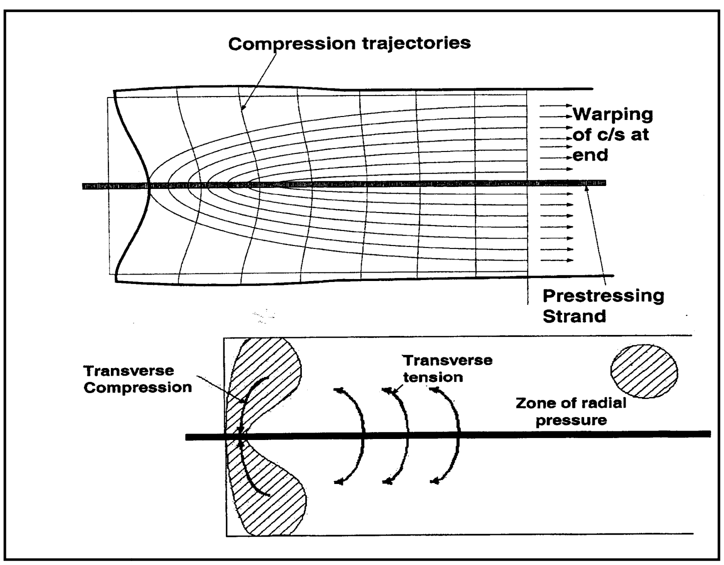


Figure 2.17: Section Warping and End Forces (M .Haq, 2005)

To summarize the “Hoyer Effect” is the greatest contributing mechanism to “bond” at the time of pre-stress release. Mechanical interlock is the main contributor to the “bond” when the stress in the strand is increased above the initial transfer stresses i.e. when the concrete cracks and the strands stress levels are increased over their initial state. Lastly, the adhesion mechanism is the smallest contributor to developing bond stresses between the strand and the concrete.

Flexural Bond Stresses:

In the design of pre-tensioned beam, the tension in the strand must increase to resist the applied moments. According to Russell (1992), as loads increase and the concrete cracks, the strands are required to carry even greater tension. The additional strand tension must be resisted by the bond stresses. These bond stresses that resist the external loads have been called “flexural bond stresses”. Before the appearance of the cracks the bond action actually plays a minor role as the shear transmission takes place just as in a section made up of a homogeneous material. Thus, the pre-tensioned beam can adequately carry working loads even without the presence of bond. If the bond does exist, then only a very slight shear stress occurs between the tendon and the concrete due to the connection between the small area of steel (times the material modular ratio) and the rest of the total cross-section. To conclude, when the working load is exceeded and cracks occur in the tensile zone of the concrete does bond become necessary.

As the concrete cracks, tension in the steel increases suddenly and abruptly. Large increases in the strand tension must be matched by large increases in bond stresses adjacent to the crack as shown in the diagram below:

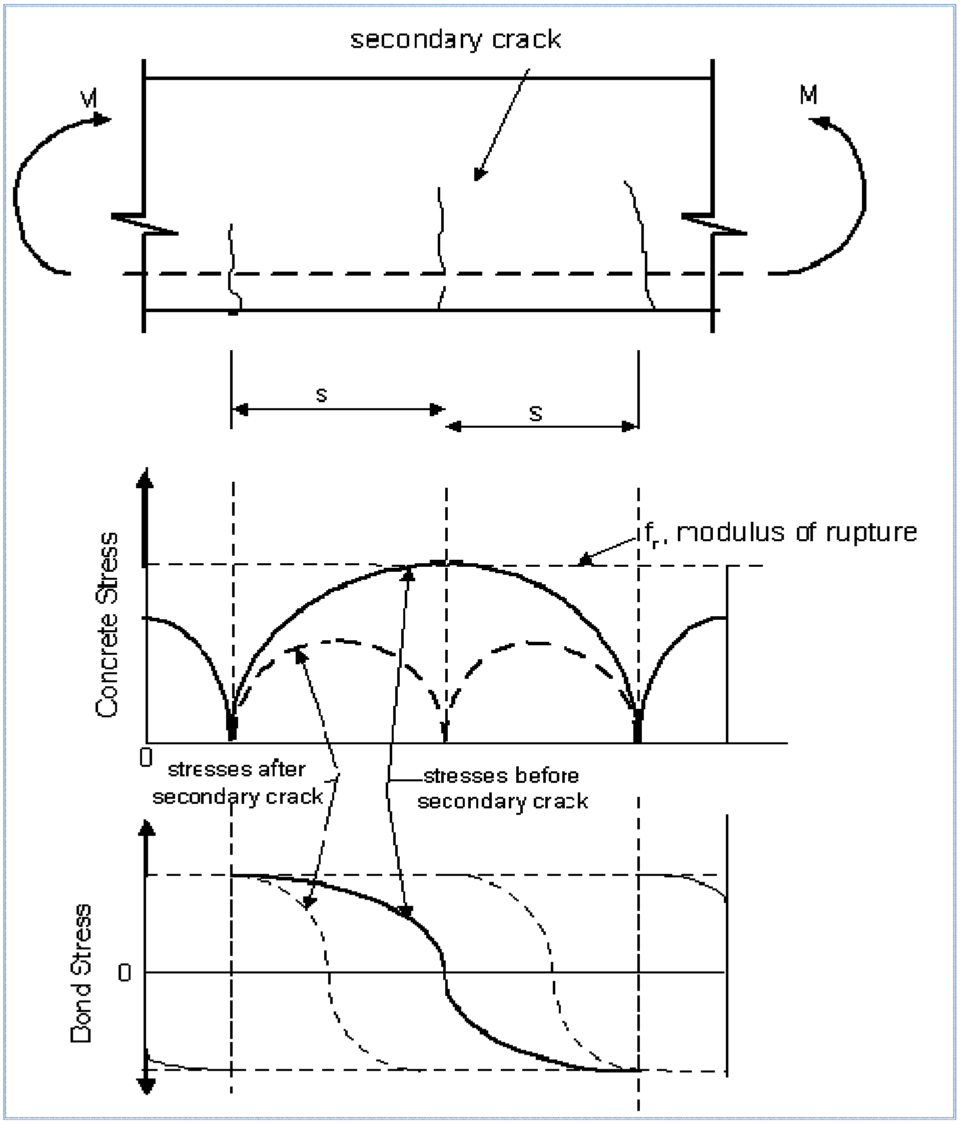


Figure 2.18: Concrete and Bond Stresses at Crack (Russell,1992).

According to Russell (1992), when a crack forms in the concrete, the strands must slip for some finite distance on either side of the crack. The length of the slip is dependent on the value of the bond stresses adjacent to the crack and the total strand slip must equal the width of the crack. The relative displacement $U_{s,}$ summed over the length of the slip equals the crack width:

$$\mathbf{(\Sigma\ Us*dl)_{both\ sides} = Crack\ Width\ (Russell\ ,\ 1992)}$$

As the bond stresses resist steel tension, they also induce tension into the concrete. As the concrete tension increases between primary cracks, the tensile strength of the concrete may be exceeded and a secondary crack may form. As shown in the diagram, the bond stresses are the highest in the immediate adjacent to the cracks and the decrease with distance away from the cracks. At the crack locations, the concrete stresses are zero. The equilibrium between the bond stresses and the concrete tension must be satisfied:

Bond Stresses*Bond Area = Concrete Tension (Russell, 1992)

Bond stresses are assumed to vary as a sine wave between the crack locations. This distribution satisfied the boundary conditions and provides a continuous function between cracks. The area of concrete tension is basically the area of the cross section immediately influenced by the strands. Equilibrium between the bond stresses and the concrete tension must be balanced. The bond stresses can be evaluated by integrating over the bonded length times the perimeter of all the strands, then setting it equal to the concrete tension:

$$\mathbf{N * P_{ps} \int u_{max} * \cos \pi/2s * dx = f_r * A_c}$$

Where,

N = Number of strands

P_{ps} = Strand Perimeter

S = Crack Spacing

f_r = Modulus of the concrete

A_c = Area of concrete that resists tension

U_{max}= Maximum Bond Stresses.

However, the development of new flexural cracks towards the end of the member will continue with the increasing strength demands due to the load distribution. The bond stress demands will follow along with the localized high stress demands at the crack. Increased tensile stresses in the strand will cause a reduction of the cross-sectional area due to Poisson's effect. Thus, if the cracking extends into the transfer zone region, the reduced cross-section tendon area will compromise the Hoyer effect, which is the main mechanism for the bond in the transfer region. The relative slip of the strand can occur leading to a reduction in the pre-stressing force and thus limiting the attainment of the section full flexural capacity. In addition, the reduced compression state at the beam end will decrease the section shear capacity

2.5 Concluding Remarks:

Moreover, the research performed on the structural significance of pre-stress bond with the high strength concrete had provided us with sufficient evidence that HSC had a significant effect on the bond strength of the strands. The high strength concrete produces higher bond strength (Eden, 2006). However, with the ongoing advent of self-consolidating concrete and its application in the pre-stress industry definitely adds further attention to the effect of bond.

Thus, in this research the size and the coarse aggregate content are the variables that are used for developing the SCC mix designs. After, performing the fresh properties like Slump, J-Ring, L-Box and V- funnel the mix designs were tested for pre-stress bond.

CHAPTER III

Mix Design Development and Evaluation

3.1 Concept of mixing process of SCC for bond test:

In order to achieve a well balance between three major attributes of SCC i.e., “segregation resistance”, “filling ability” and “passing ability” a very prudent and extremely accurate mixture proportioning is to be done. SCC is more sensitive to any departure from the target mix design and mixing technique than the conventional concrete. This means that the allowable deviations in weighing and dispensing are smaller, the allowable variations in the concrete constituents (e.g. variations in aggregate shape and grading curves, moisture content, cement composition) are smaller, and batching sequence and mixing times should be followed more closely. Thus, a greater diligence should be executed in documenting concrete and raw material properties.

The flow ability of a concrete mix is a complex interaction between the inter-particle friction in the in the aggregate phase and the fluidity of the paste phase. The water-to-powder ratio and the admixtures control the fluidity of the cement paste. If the aggregate particles have too much friction due to poor grading or shape, the paste will have to be very fluid to compensate and achieve the desired concrete flow-ability. If the paste is too fluid, segregation will result. Thus to start with it is always recommended to select the most consistent and best-graded and shaped aggregate economically possible, and to use high cement paste fractions to increase space between the aggregate particles. Estimating the required batch weights involves a series of steps that yields a trial batch with the desired rheological properties of SCC.

3.a) Determining Required slump flow:

Before starting the proportioning of SCC, it is important to take into consideration the project specifications. The review will guide us in attaining the slump flow, compressive strengths and age when the strength is to be attained.

Therefore, the proportioning was adopted from the PCI Interim Guidelines for the Use of SCC in PCI Member Plants and other referral documents like EB001(Design and Control of Concrete Mixtures), ACI 211.1-Standard Practices for Selecting Proportions for Normal, Heavyweight and Mass Concrete and ACI 301-Specifications for Structural Concrete for Buildings (ACI).

The mixes shall be evaluated by trial batches prepared in accordance with ASTM C 192 and production tests under conditions simulating as closely as possible actual production and finishing. Each SCC mix water should be evaluated to assure its ability to accept the water dosage variation consistent with the ability of the plant equipment to control total mix water without showing excessive bleeding or segregation. Thus the w/cm (water to cementitious) material ratio is one of the fundamental keys governing the strength and durability of the concrete.

3.b) Coarse and Fine Aggregate Selection and Proportion:

The nominal maximum size of the coarse aggregate must be chosen with respect to obtaining the desired passing ability and stability of the plastic concrete. Blending always proves to be beneficial when working with two or more different aggregate sizes to obtain an optimum gradation. The particle shape of the coarse aggregate can have a significant impact on the performance of a SCC mix. A rounded coarse aggregate will impart greater filling ability to a mixture when compared to a crushed stone of similar size. All other parameters being equal, a higher volume of well rounded natural aggregate could be used in a concrete mix than of an angular crushed aggregate having the same gradation. Unlike the conventional concrete, when adjusting the proportions of a SCC mixture to achieve proper yield , all the constituent aggregates should be adjusted simultaneously so that the overall gradation of the aggregates is not affected. The quantity is determined by using the dry rodded bulk density calculations as per the Portland Cement Association, Table 9-4 and 9-5. However, it was found that for normal density aggregates this typically yields an absolute volume that is 28-32 % of the concrete volume, with the remaining 68-72% being mortar. Thus, to conclude the size, gradation and surface texture will influence the volume of the coarse aggregate that will permit acceptable passing and filling ability of the plastic SCC. The highly gap graded aggregate mixtures was avoided as the SCC mix will have a tendency to bleed and segregate and will increase the overall paste fraction requirements of the concrete.

3. c) Powder and Water Content:

The powder includes Portland cement, supplementary cementitious materials and inert fillers passing a No.100 sieve. The inert fillers, obtained by grinding calcareous or siliceous aggregates can be achieved by better packing density. The fine fraction of these fillers will increase the specific surface of the blend, while the coarser fractions can help bridge the gap between sand and the Portland cement. However, for the mix designs used here and for the bond test no such fillers was used.

The fineness and the volume of the powder, in conjunction with the fine aggregate, help from a mortar mix that supports the coarse aggregate. When performing the trial batches it was prudent to start with high powder contents, and then optimizes the mix for improved economy. It is very important to maintain the consistency, between the batches of SCC and all the high quality concrete, requires diligent monitoring and adjustment for the aggregate free surface moisture. As there were no in line aggregate moisture meters to detect the moisture content and the moisture variations in the SCC mixes, the free moisture of all aggregates shall be determined at the beginning of each batching operation and at a 4 hr intervals during continuous batching operations. The samples for moisture determination was taken from aggregate that is represents the actual aggregate going to be placed in the mixer.

3. d)Admixture Selection:

The poly-carboxylate HRWR admixtures are used in developing and proportion SCC mixtures. The viscosity modifying admixtures (VMA) are beneficial for adjusting the viscosity and may be used to improve the stability of SCC mixtures. It is advantageous when using lower powder contents as well as when using the gap graded, angular, and elongated aggregates.

In functional terms, the HRWR's impart the fluidity to the SCC mixture, while VMAs (like VMAR3) provides an increase in viscosity and cohesiveness to improve the mixture's stability. Thus, the use of a VMA in conjunction with a HRWR may also increase the water tolerance of a mixture.

To conclude, after gaining the above fundamental building blocks of a SCC mixture we developed three SCC mixes with variables as the coarse aggregate size and content with the fixed dosages of HRWRA and VMA's.

3.2 Project Mix Design Matrix For Bond Test:

3.2 a) Cement:

The cement used for the conventional concrete, self consolidating concrete and mortar testing program was Type II cement from Unicem USA . The Portland Type 3 cement used conformed to ASTM C 150. The detailed chemical analysis of the cement used is reported in Appendix . The Type 3 cements used for the STSB tests are stacked in wooden pallots and covered with plastic sheets to control the moisture content of the cement.

3.2 b) Aggregates:

The fine aggregates used for the NASP testing was supplied by the Dolese Brothers from their plants in Guthrie, OK . The fine aggregates used was natural quartz and comply with ASTM C 33 specifications. It had a specific gravity of 2.61 and absorption content of 0.489%. The fineness modulus of the fine aggregates used was 2.81.

Similarly, the coarse aggregates all conform to ASTM C 33 specifications. The maximum size of the aggregates depends on the particular applications and mostly limited to ¾ in size. It was also supplied by the Dolese Brothers from their Stillwater Plants. The two sizes that were used in the research 3/8 in and 3/8 in, so that the fresh properties and the hardened properties of SCC can be evaluated in terms of their size and content. The 3/8 in had a specific gravity of 2.69 and absorption content of 3.22 %. The ¾ in rock had specific gravity of 2.61 and absorption content of 1.78%. The gradation and other properties of the C.A are as per ASTM C 127, C 702 and C 566 are shown in the table below:

Table 3.1 : Sieve Analysis of C.A used for STSB test:						
Sieve Number	Empty Sieve Wt (gm)	Sieve and Sample Wt (gm):	Rock Wt (gm):	% Retained	% Passing	Cum % Retained
1 in	7268.7	7268.7	0	0	0	0
3/4 in	7265.8	7730.4	464.6	8.35	91.65	8.35
1/2 in	7134.7	10769.7	3635	65.34	26.31	73.69
3/8 in	7138.4	8136.4	998	17.94	8.37	91.63
No. 4	6942.8	7304.7	361.9	6.51	1.86	98.14
Pan	6054.8	6158.3	103.5	1.86	-	-

3.2 c) Mixing Water:

The mixing water used for the bond tests was free from deleterious matter that might had interfered with the colour, setting or strength of the concrete. As the water is potable, it is used to make concrete.

3.2 d) Admixtures:

The trial mixture program demonstrated satisfactory performance of the admixture relative to the SCC fluidity, stability, workability, air content and strength requirements under the conditions of use, particularly with respect to temperature and humidity typical of production conditions. The admixtures used in the SCC were carefully checked for the compatibility with the cement and other admixtures to ensure that each performs as required without affecting the performance of other admixtures. Thus , by trial mixing with a range of cement contents –w/c ratios, slump flow values and other properties like segregation, passing ability, filling ability the effect of variations in the dosage and sequence of charging the admixtures into the mixer was determined .

Three main admixtures that were used in the bond test research were ADVA CAST 575 (a high range water reducing admixture), V MAR 3 (a viscosity modifying admixture or a concrete rheology modifying admixture) and RECOVER (a hydration stabilizer). All these admixtures were supplied by Grace Chemicals. A brief overview of these admixtures will help us in understanding the mix designs performed later for the bond test.

a) ADVA CAST 575 :

It is a high efficiency; low addition rate poly-carboxylate based high range water reducer designed for the production of SCC mixes. It meets the provisional requirements of ASTM C 494 as a Type A and F, and ASTM C 1017 Type 1 plasticizing. It is supplied as a ready to use liquid that weighs approximately 8.9 lbs/gal. It does not contain intentionally added chlorides. It is formulated to impart improved workability, stability, tolerance to the concrete even at low water/cementitious materials and also sometimes to attain high early strength as required by the precast industry.

Addition Rates:

The dosage requirements were adjusted as per the wide spectrum of the concrete performance requirements. The addition rates can vary from 2 to 10 fl oz/100lbs of the cementitous, but typically ranges from 3 to 6 floz/100lbs depending on the mix designs, cement content, aggregate gradations, and ambient conditions. The ADVA CAST 575 was added with half the mixing water at the middle of the batching sequence. It was taken proper care that it did not come in contact with other admixtures before they enter the concrete mixer. The Material Safety and Data Sheet and other properties is attached in Appendix.

b) VMAR 3:

It is a high efficiency, liquid admixture designed to enable the production of SCC by modifying the rheology of the concrete. V-MAR 3 works by increasing the viscosity of the concrete thereby still allowing the concrete to flow without segregation. It has a unique polymeric structure that, under the influence of energy (e.g. vibration or pumping), aligns itself and allows coarse angular sand to flow similarly to the naturally rounded sand. At this point the polymers slide over each other in the direction of the flow and reduce the yield stress of the concrete. As the energy is removed, the polymers interlock leaving the concrete as it was before movement.

Addition Rates:

It is typically used at an addition rates of 10 to 40 fl oz/yd^3 of the concrete. As the water content increases, the V MAR 3 requirements will increase. At lower water contents, the use of V MAR 3 at the lower dosage range was used. However, it is also affected by the mix design, cementitous content and aggregate gradations. It worked well with the ADVA class of super-plasticizers. It was added in to the mix as a part of the mixing water.

c) RECOVER :

Recover is a ready to use aqueous solution of chemical compounds specifically designed to stabilize the hydration of Portland Cement concretes. It is a ASTM C 494 Type D Retarder. It is used as a controlled set time extensions was required as per the need of the project. The usage was determined from the trial mixing requirements. The time needed to evaluate all the fresh properties of SCC was calculated (as per the project specifications) and then the dosage was proportioned as per that. It was compatable with most of the Grace admixtures, as long as they are added separately into the concrete mixer. It was added at the end of the batching sequence for optimum performance.

Addition Rates:

For traditional cases, the dosages may vary from 5 to 50 floz/100 lbs of the cementitous or 2 to 6 fl oz/100 lbs of the cement. Thus , the actual dosage was fixed by trial batching and pre-testing.

3.2 e) Pre-stressing Strands:

All the pre-stressing strands used for this research program were seven wire 270 ksi Low relaxation strands from manufacturers in North America. The strands conformed to ASTM 416 specifications for Low relaxation strands as attached in Appendix .The pre-stressing strands had a nominal diameter of 0.5 in (12.7 mm). The nominal cross-sectional area was 0.153 sq in (98.7 mm^2) for 0.5 in diameter strands. The modulus of elasticity of the pre-stressing strands was estimated as 28,500 ksi (196.3 Gpa).

3.2 f) Trial Mixes:

The concrete mixtures used for making NASP specimens in concrete included the Type 3 cement from Buzzi Unicem USA, IN and fine aggregate from Dolese Brothers Co. and admixtures from Grace Chemicals Inc. The admixtures used were High Range Water Reducers specifically used to make SCC, Viscosity Modifying Admixtures and a very little amount of Set Retarders. The HRWRA used for making SCC was ADVA CAST 575. The VMA used was V MAR 3 and a little amount of RECOVER (hydration stabilizer) as per the need of the fresh properties of the mix. The mix proportions were designed based on the target one day strength in the range of 4500 to 5000 psi as specified for STSB test. The mix proportions C-N, SCC -1, SCC -2 and SCC -3 were all designed to meet the target strength of

4750 to 5000 psi in order to account for the bonding ability of pre-stressing strands. Various w/c ratios were tried like 0.44, 0.47 and 0.49. Then the results were interpolated to achieve a final w/c ratio of 0.46. However, while performing the SCC trial batching it was found that the HRWRA and VMA should be made the part of the mixing water in order to avoid clumps as soon as it hits the cement. The improper mixing is due to the incomplete hydration of the cement particles. The fresh properties like the slump flow, J-Ring and L – box were repeated for every single trial batches in order to ensure the stability of the SCC mixes. Every time the results proved that the mix was stable enough to further work on other variables. The main purpose of the research was to find whether by changing the coarse rock content and size of the rock had any effect on the stability, filling ability and passing ability of the SCC and testing those mixes against pre-stress bond, keeping the strength factor same. Then the ideal mixes satisfying all the requirements of strength and fresh properties were tested for pre-stress bond.

C-N (Normal Concrete):

Controlling w/c (0.46)		Air Content	2%	for 3/4"	0.64
Mix Component		Weight(lb /CY^3)	Sp Gravity	Density of water (lb /ft^3)	Absolute Vol (ft^3)
Adjusted Water		298	1	62.4	4.8
Total Cement					
Portland Cement		648	3.15	62.4	3.3
Recover		-	1	62.4	0.0
High Range WRA		-	1	62.4	0.0
VMA		-	1	62.4	0.0
Total Paste Volume					
Total Air Content		0			1.6
Air + Paste Vol.		-	-	-	
Total Agg. Volume		-	-	-	
Aggregate Data					
Coarse Aggregate		1722	2.68	62.4	10.3
Intermediate Agg		N.A	N.A	N.A	N.A
Fine Agg		1146	2.612	62.4	7.0
Total Weight /CY		3814			
Total Absolute Volume					27.0

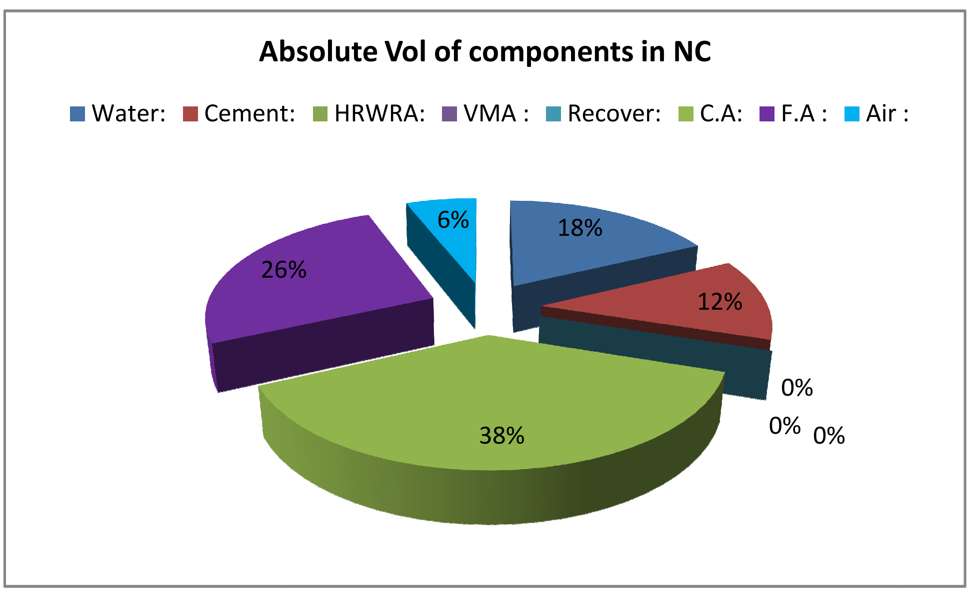
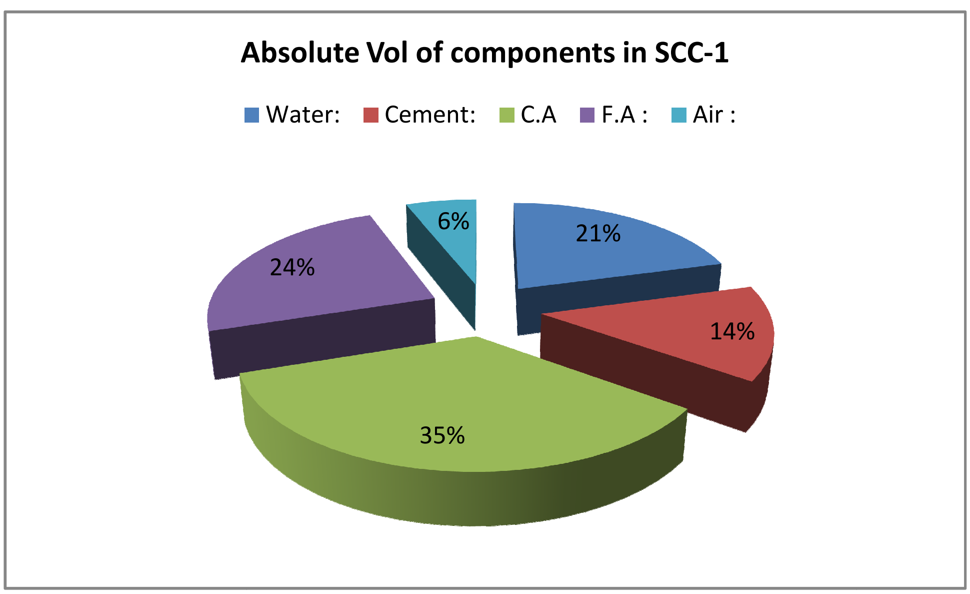


Figure 2.19: Pie – Diagram Showing the Components in Normal Concrete (NC)

Based on the above C-N, we reduced the rock content and increased the fines to obtain the SCC -1. The admixtures were proportioned as per need of the project specifications.

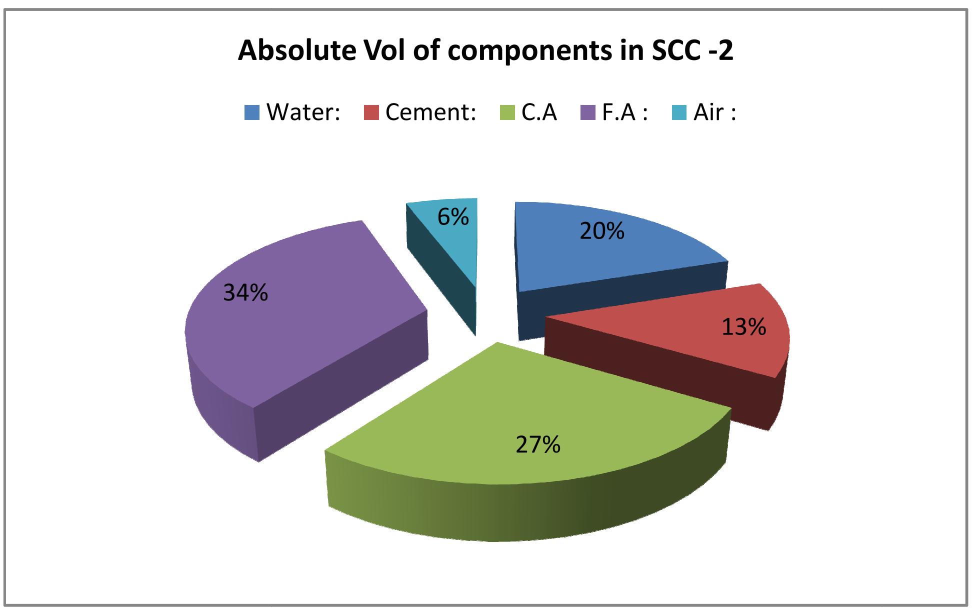
SCC -1

w/c (0.47)		Air Content	2%	For 3/4"Rock	0.64
Mix Component		Weight(lb/CY^3)	Sp Gravity	Density of water (lb /ft^3)	Absolute Volume (ft^3)
Adjusted Water		350	1	62.4	5.6
Total Cementitious					
Portland Cement		745	3.15	62.4	3.8
Recover (fl oz)		1	1	62.4	0.1
HRWRA (fl oz)		10	1	62.4	1.2
VMA (fl oz)		30	1	62.4	0.48
Total Paste Volume					
Total Air Content		0			1.6
Air + Paste Vol					
Total Agg Volume					
Agg Data					
Coarse Agg		1581	2.68	62.4	9.5
Intermediate Agg		N.A	N.A	N.A	N.A
Fine Agg		1128	2.612	62.4	6.5
Total Weight /CY		3803			
Total Absolute Vol					27.0



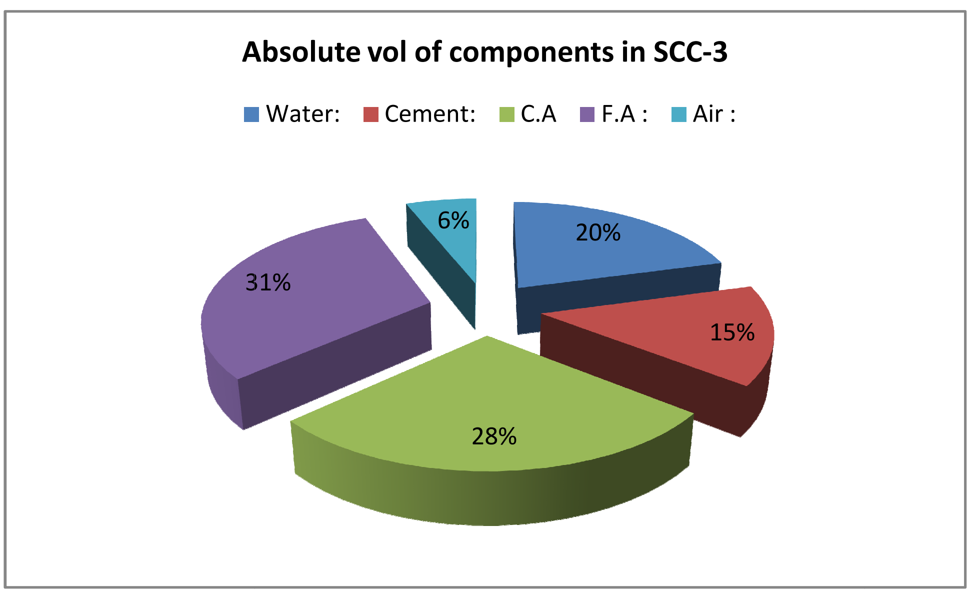
The only variable here was the reduced rock content.

w/c (0.47) :		Air Content 2%	for 3/4"		0.64
Mix Component		Weight(lb /CY^3)	Sp Gravity	Density of water (lb /ft^3)	Absolute Volume (ft^3)
Adjusted Water		350	1	62.4	5.6
Total Cementitious					
Portland Cement		745	3.15	62.4	3.8
Recover (fl oz)		1	1	62.4	0.1
HRWRA (fl oz)		10	1	62.4	1.2
VMA (fl oz)		30	1	62.4	3.6
Total Paste Volume					
Total Air Content		0			0.5
Air + Paste Volume					
Total Agg Volume					
Aggregate Data					
Coarse Agg		1264	2.68	62.4	7.6
Intermediate Agg		N.A	N.A	N.A	N.A
Fine Aggregate		1549	2.612	62.4	9.5
Total Weight /CY		3908			
Total Absolute Vol					27.0



The dosages of admixtures were kept same for all the mixes as that was not the variable for the bond test.

w/c (0.45)		Air Content	2%	for 3/8"	0.48
Mix Component		Weight(lb /CY^3)	Sp Gravity	Density of water (lb /ft^3)	Absolute Vol (ft^3)
Adjusted Water		350	1	62.4	5.6
Total Cementitious					
Portland Cement		778	3.15	62.4	4.0
Recover (fl oz)		1	1	62.4	0.1
HRWRA (fl oz)		10	1	62.4	1.3
VMA (fl oz)		30	1	62.4	3.7
Total Paste Volume					
Total Air Content		0			1.6
Air + Paste Vol					
Total Agg Volume					
Agg Data					
Coarse Agg		1257	2.68	62.4	7.5
Intermediate Agg		N.A	N.A	N.A	N.A
Fine Agg		1393	2.612	62.4	8.3
Total Weight /CY		3778			
Total Absolute Vol					27.0



3.3 Evaluation of Fresh SCC Properties:

It is important to note that none of the test methods of SCC has yet been standardised in North America. It is anticipated that the ASTM and ACI committees will develop consensus standards for SCC testing in the near future. All the fresh properties performed here are based on the Interim Guidelines for Use of SCC published by the PCI for their member plants.

3.3 a) Slump Flow Test and VSI:

1. Introduction:

The slump flow was used to access the horizontal free flow of SCC in the absence of obstructions. The diameter of the concrete circle, in the Figure 3.1, is a measure for the flow-ability of the SCC.

2. Assessment of Test:

The slump flow test is a simple and rapid test to perform. It gives a good assessment of the filling ability, but it does not give any indication of the ability of SCC to pass between the reinforcements without blocking, but may give some indication of the resistance to segregation.

The slump cone can also be used in the inverted position to perform the slump flow test. The values of the slump are nearly the same as determined by either the upright or the inverted slump cone. Below shows a diagrammatic representation of the test :

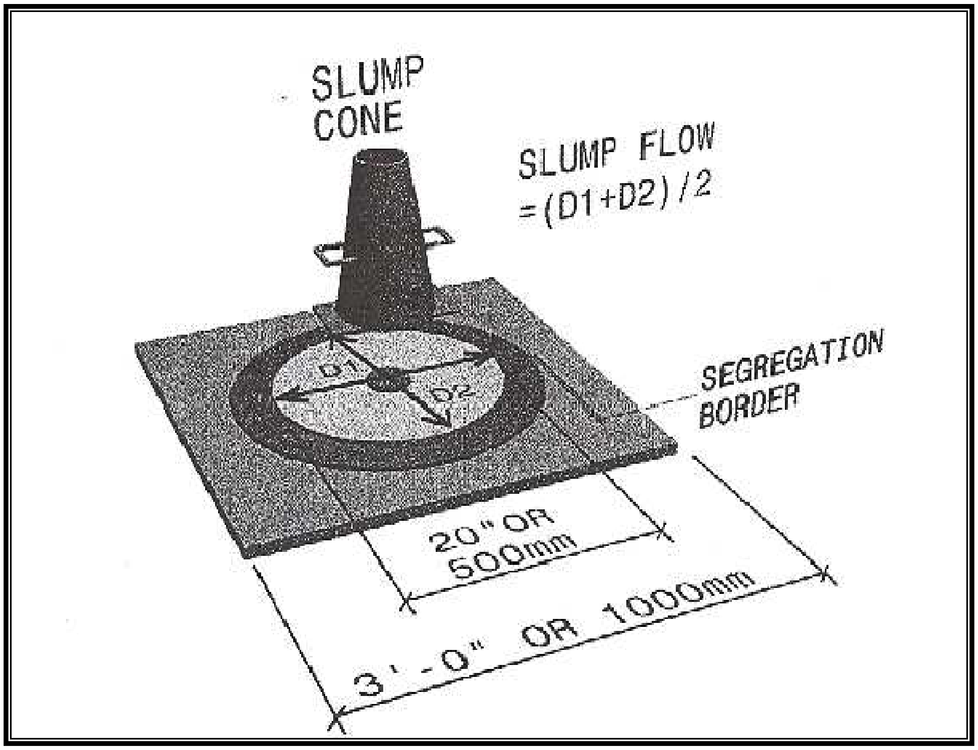


Figure 3.1: Slump Flow Test (PCI, Interim Guidelines)

3. Equipment:

The apparatus used for both the Slump and Inverted Slump are as follows:

a) The mould in the shape of a truncated cone with the internal dimensions 8 in at the base, 4 in diameter at the top and a height of 12 in.

b) The Base Plate was made of a stiff non-absorbing material, at least 28 in sq marked outside and a further concentric circle of 20 in diameter.

c) Trowel

d) Scoop

e) Ruler to measure the spread.

4. Procedure:

The process of measuring the slump flow both by normal and inverted were same as follows:

a) The cone was filled with SCC, sampled normally.

b) The base plate and the inside of the cone were moistened.

c.) The base plate was placed on a level ground and the slump cone was centrally placed on the base plate and held firmly.

d) Then the cone was filled with the scoop. Without tamping, the SCC level was simply stroke off from the top of the cone with a trowel.

e) The surplus SCC was removed from the base of the cone before lifting, so that the flow should be free from any kind of hindrances.

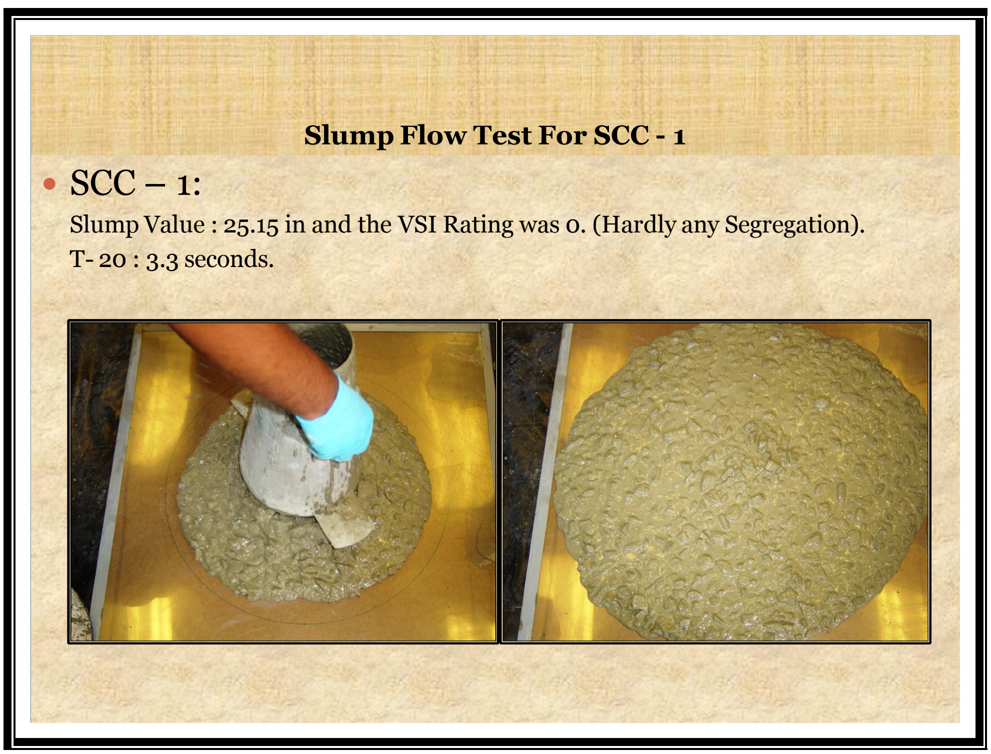


Figure 3.2: Showing the procedure of measuring Slump flow for SCC-1, less rock than NC**.**

f) Then the cone was raised vertically and allowed the SCC to flow freely.

g) Then the final diameter of the SCC was measured in two perpendicular directions.

h) The average of the two measured diameters was calculated and the slump flow was reported in inches.

i) The rate of the stability of the mixture in 0.5 increments was done by visual examination as prescribed by the PCI guidelines.

j) Simultaneously, the T-20 inch test was also performed to measure the filling ability of the SCC mixture, which is also a measure of the mixture viscosity. The stop watch was started once the cone was lifted and then stopped once the flow hit the border mark.

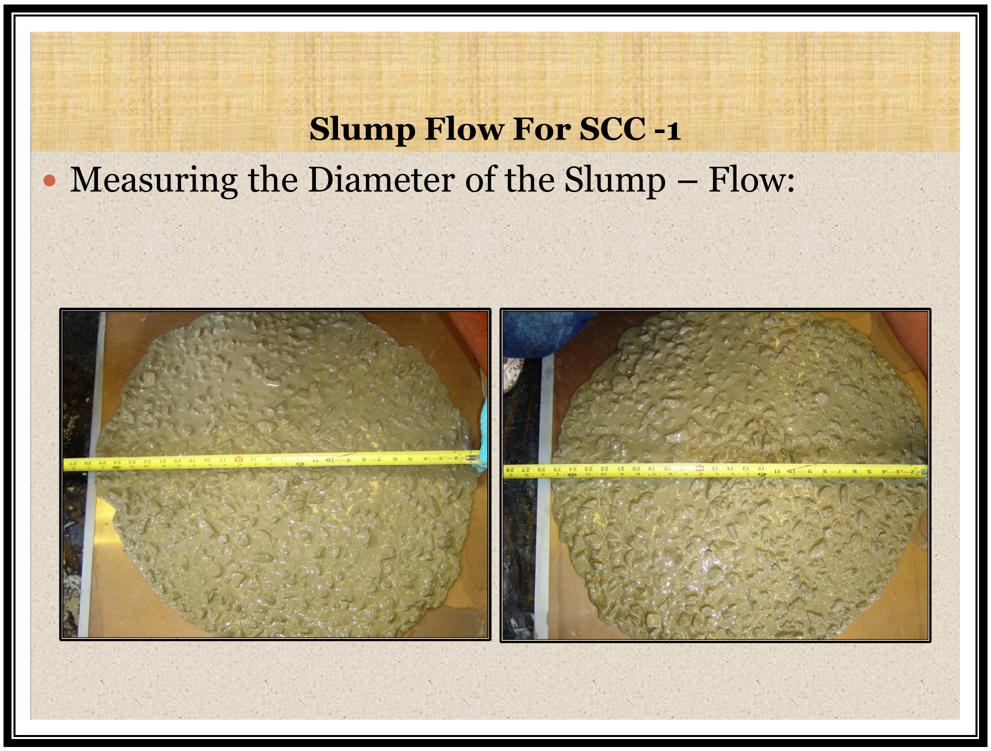


Figure 3.3: Showing the measurement of slump in two mutually perpendicular directions.

l) The T-20 time measurements are in the range of 2 to 7 seconds, the main point of the slump flow test method is not the speed in which the concrete flowed but the diameter of the spread achieved by the concrete under the effects of gravity only.

m) The T20 is a secondary indication of the flow. The test indicated a possible deviation in the production and can help in raising the quality control of the mixes.

n) As the cone was lifted, the time taken by the SCC mix to reach the 20 in spread circle was recorded as the T20 time for the mix**.**

Figure 3.3: Slump Flow For SCC-2

5. **Observation:**

The SCC-1 , SCC-2 and SCC-3 have a measured slump flow of 23.15 in, 27.1 and 26.15 in respectively. The VSI rating for the mixes were as 0 , 0.5 and 0 respectively. This means that SCC-1 and SCC-3 are good mixes and showed no sign of segregation either in the form of mortar patty, aggregate pile on the top of the mix. However, for SCC-2 , a little bit of segregation was there due to the higher content of the fines and lesser coarse aggregate, there was a tendency of the mix to loosen apart due to improper gradation. This was taken care by appropriate dosage of VMA in conjunction with HRWRA and strict moisture control at the time of trial batching.

Figure 3.4: Slump flow on SCC-3

Inverted Slump Flow Test:

The test was similar to the slump flow test with the only difference is the cone held in the inverted position. This was also a good assessment to the flow-ability of the mix. In the severe segregation, most of the coarse aggregate would be in the centre of the pool of the SCC mix. In the case of minor segregation a border of mortar without the coarse aggregate can occur at the edge of the pool.

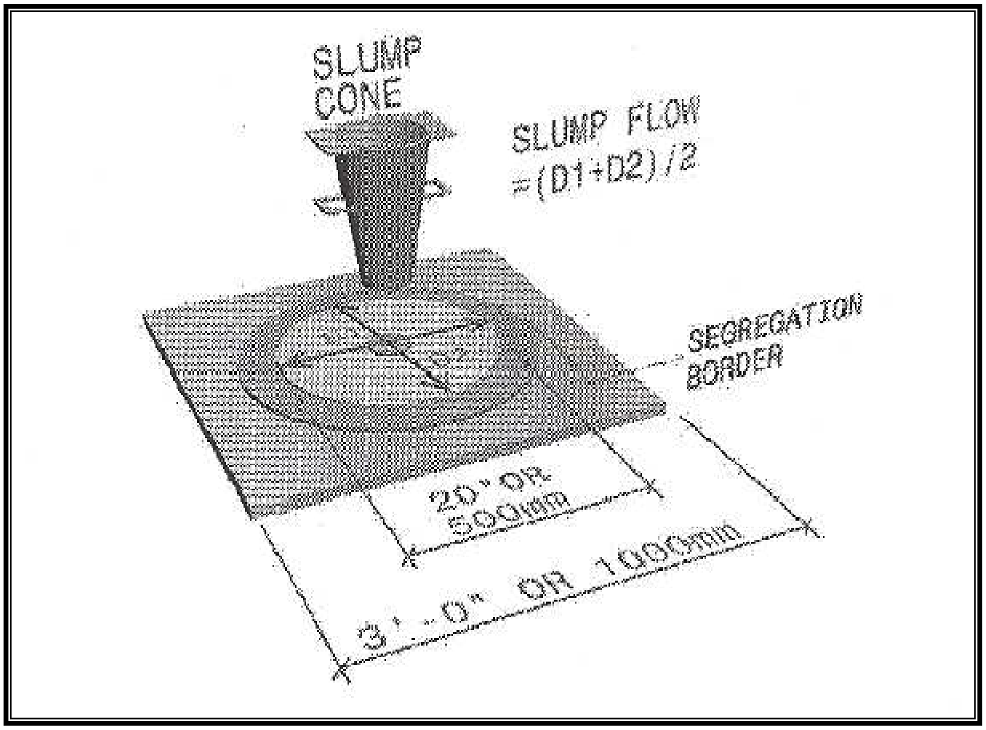


Figure 3.5: Showing the Inverted Slump Flow.

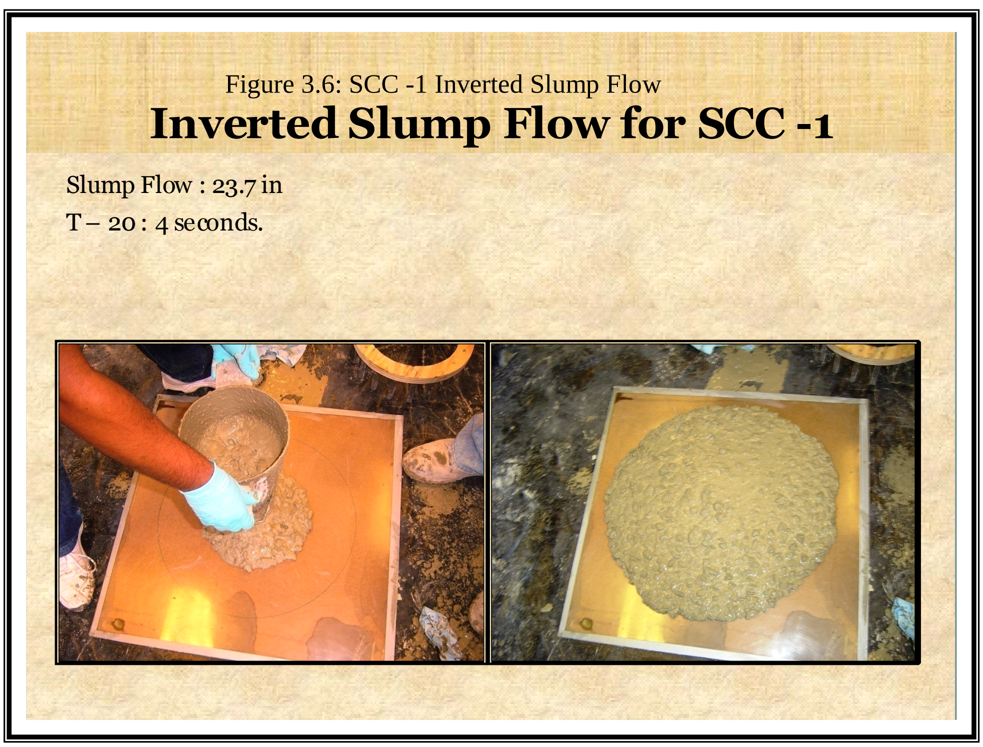


Figure 3.6: SCC -1 Inverted Slump Flow

Figure 3.7: Showing the IS flow for SCC-1, with reduced rock content.

Figure 3.8: IS flow test for SCC -2, the effect of reduced rock size on flow.

There was a little segregation in the mix as observed but it was accepted as per the VSI guidelines of the PCI. As the mortar fraction was more, the coarse aggregate had a tendency to separate apart from the rest of the mix, but the VMA helped in reducing the segregation. The T-20, measured was 4 seconds which was very well within the accepted range of the PCI. Similarly, the inverted slump flow was tested for SCC-3. The mix was made up of smaller coarse aggregate size, which gave a proper gradation and segregation was the least. The mix behaved very much like the cohesive mix. Also, the T-20 measured was within the range as per specified by the PCI.

Figure 3.9: Inverted Slump Flow for SCC- 3

6. Interpretation of the Results:

During the SCC mixing the parameters that were varied were the coarse aggregate size and its content. The SCC had a more paste content due to its higher fraction of the fines as compared to the normal concrete. The inherent qualities of SCC which gave its unique feature like the filling ability in congested reinforcements is the higher percentage of fine to coarse aggregates and increased cementitious. The slump flow does not give the true representation of the filling ability as for this other tests need to be performed. Also, it showed only the qualification of the mix as a "SCC" mix in conjunction with the T20 test. The SCC-1 had a slump of 25.15 in, SCC-2 had a slump of 27 in and SCC -3 had a slump of 26.15 in. The F.A/C.A was increased in the mixes was increased gradually from 71.3%, 81.6% and 90.2% for SCC-1, SCC-2 and SCC-3 respectively. The increase in the fines had hardly any effect in the slump flow as the major contributing factor was the dosages of HRWRA and VMA which were kept constant during all the mixes. However, the T20 timing were 3.3 seconds, 2 seconds and 2 seconds. The flow-abilty had some effect on the fines as the time taken to flow was reduced due to lesser blockage of C.A.

3.3 b) J –Ring Test Methods:

1. Introduction:

The test was used to determine the passing ability of SCC. The equipment consisted of rectangular section 1-1/8 by 1 in open steel circular ring drilled vertically with the holes to accept threaded sections of the reinforcement bar. These sections of the bar can be of different diameters and spaced at different intervals: in accordance with the normal re-inforcements considerations, three times the maximum size of the aggregate might be appropriate (as per PCI)

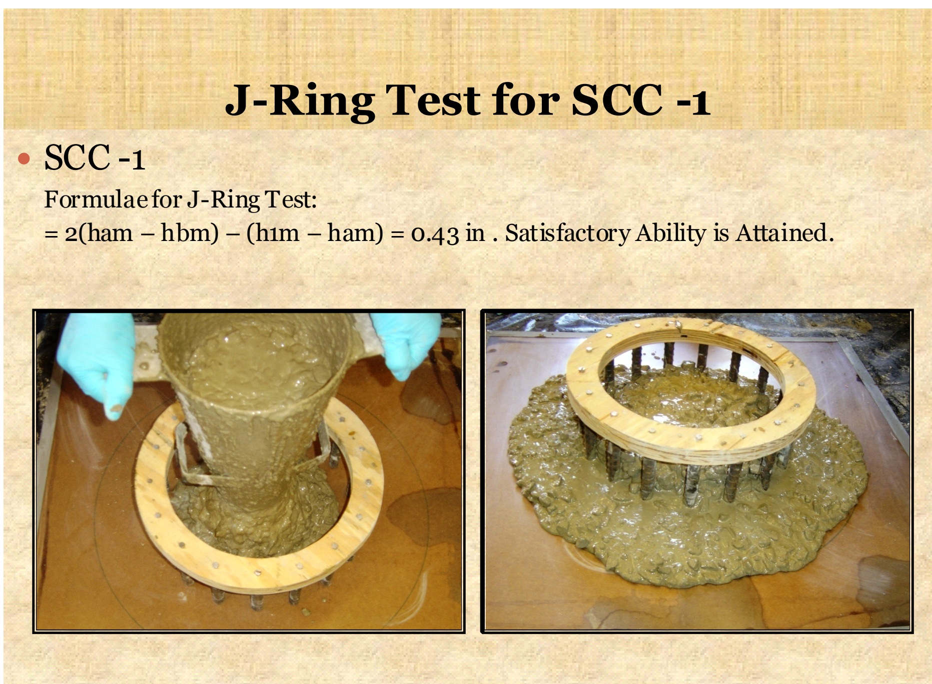


Figure 3.9: J-Ring Test on SCC -1.

The figure above shows the passing ability of SCC -1 , in the J- ring Test. This helped in finding out the passing ability of the SCC mix, when it is placed in a closely spaced reinforcements.

As per the PCI Guidelines, the number of bars in the J-Ring based on the maximum nominal coarse aggregate size and size of rebar are described as below:

Maximum Nom. Size of Aggreagte	Spacing c/c of the rebar (rebar dia 5/8")	Clear spacing between the outside of rebar	No. of rebars
8mm or 1/4"	30 mm or 1 1/8"	14mm or 1/2 "	31
10mm or 3/8"	35 mm or 1 3/8"	19 mm or 3/4"	27
20 mm or 3/4"	55 mm or 2 1/8"	39mm or 1 1/2"	17

Table 3.2: Showing the number of rebars required in J-Ring, (PCI Interim Guidelines).

2. Assessment of the Test:

After the test, the difference in the height between the SCC inside and that just outside the J-Ring was measured. This gave an indication of the passing ability, or the degree to which the passage of SCC through the bars was restricted.

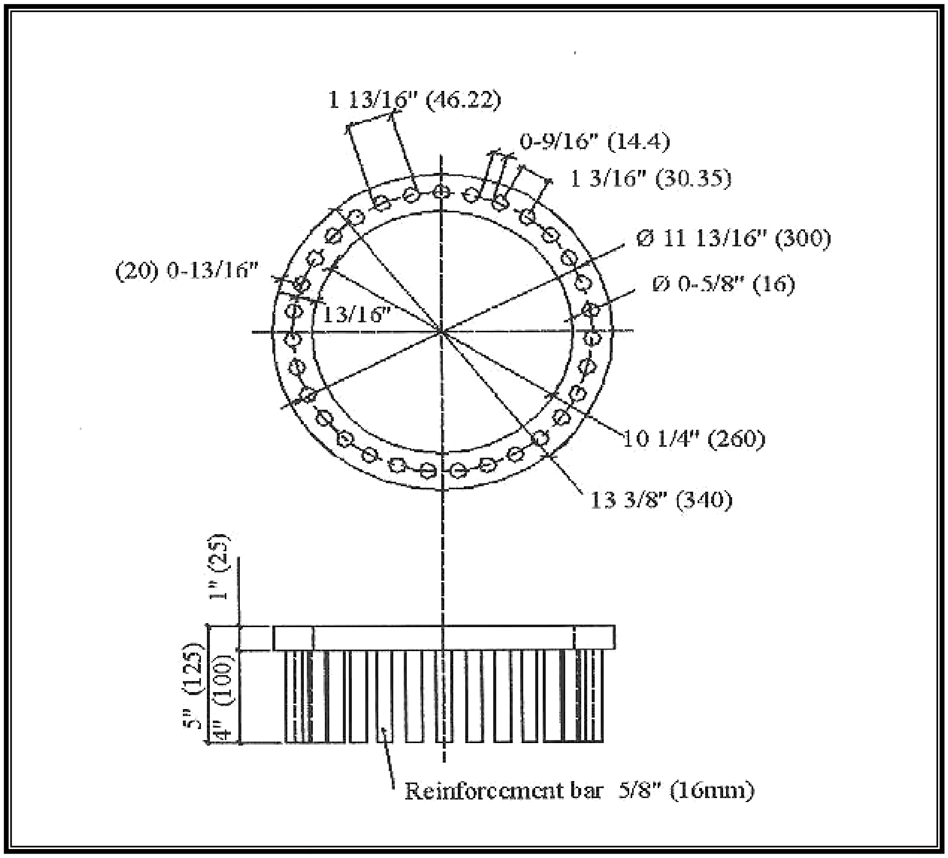


Figure 3.10: J-Ring Set up and its schematic representation.

Figure 3.11: J-Ring Test on SCC -2.

3. Equipment:

a) The mold without the foot pieces, in the shape of a truncated cone was used with the internal Dimensions 8in diameter at the base, 4 in diameter at the top and a height of 12 in.

b) The base plate of a stiff non absorbing material , at least 28 in square, marked with a Circle marking the central location for the slump cone and a further concentric circle of 20 inches diameter.

c) A Trowel

d) A Scoop

e) A Ruler to measure the height of the mix after it had passed through the reinforcements.

f) J- Ring, a rectangular section 1-1/8 by 1-inch open steel circular ring, with drilled holes vertically.

g) However, in the holes can be screwed threaded sections of the reinforcement bar length 4in, diameter 3/8 in and spacing 1-7/8+/- 1/ 8 inches (as per the PCI guidelines)

4. Procedure:

a) About 0.2 ft^3 of SCC was needed to perform the test, sampled normally.

b) The base plate and the inside of the slump cone were moistened.

c) The base plate was placed on a level stable ground.

d) The J-ring was placed centrally on the base plate with the slump cone centrally inside It and held down firmly.

Figure 3.12: J-Ring Test on SCC -3.

e) Then the cone was filled with the scoop. It was not tamped, only the SCC level was stroke off from the top of the cone with the trowel.

f) Then , the surplus SCC was removed from around the base of the cone.

g) The cone was raised vertically and the SCC was allowed to flow out freely.

h) The final diameter was measured in two perpendicular directions.

i) Then , the average of the two diameter was measured in inches.

j) The difference was measured in height between the SCC just inside the bars and that just outside the bars.

k) Then the average of the difference in height at the four locations was expressed in inches.

l) It was noted that whether there was any border of mortar or cement paste without C.A at the edge of the pool of SCC was there or not.

Calculation of J-Ring Value:

a) The value of d1 was measured in the center of the J-Ring and also 4 values da and db just inside and just outside the ring.

b) H1= 125 – d1 and all h values hax = 125 – dax (x = 1 to 4) was calculated.

c) Then the 4 values h1-hax was calculated and the median value h1m – ham.

d) Then the 4 values hax – hbx was calculated and the median value ham – hbm.

e) The J-Ring value was calculated using the formulae :

= 2(ham – hbm) – (h1m – ham) where , the designations ham, hbm and h1m are shown.

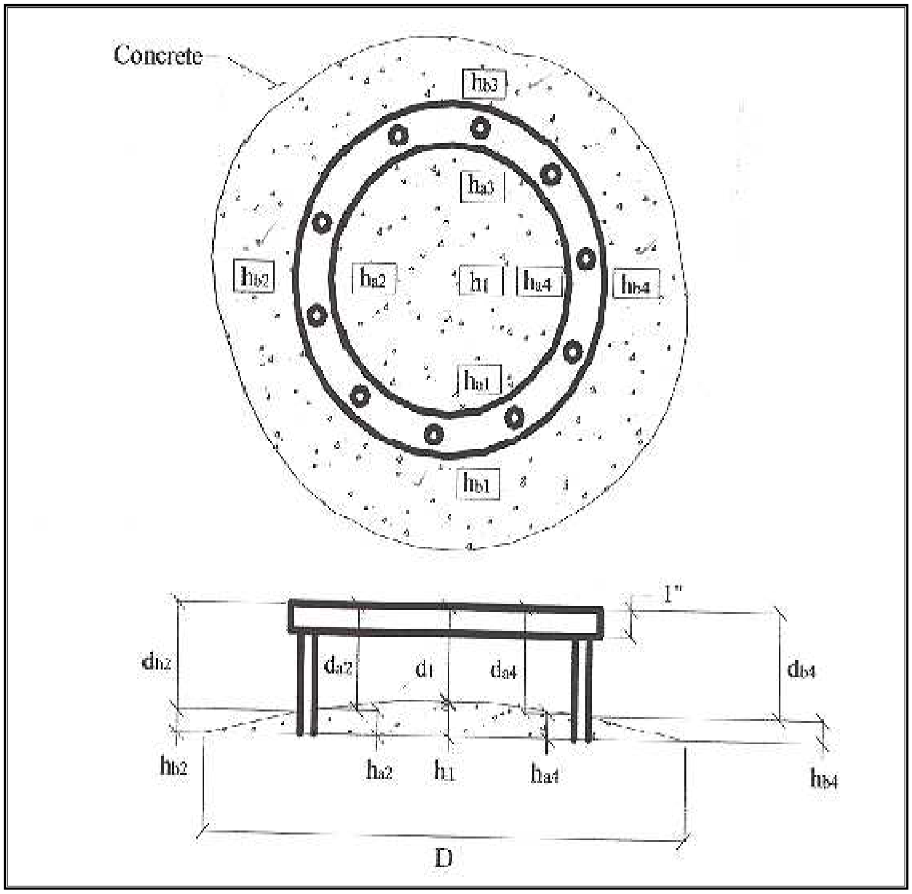


Figure 3.13: J-Ring Calculation Method (PCI Guidelines)

5. Interpretations of the Test Results:

The J-Ring values for the SCC mixes were reduced with the increase in the F.A to C.A ratios. The J-Ring values were 0.429 in, 0.129 in and 0.625 in for SCC-1, SCC-2 and SCC -3 resp.

By adding finer particles, the filling and passing ability of the mix was more. However, this is due to the fact that by increasing the binder content, there was low yield stress, lesser inter-particle collision and less blocking of the mix. This is due to the better particle packing density, resulted in better flow to viscosity ratio. This actually helped the matrix to move as a cohesive fluid resisting both the static and dynamic segregation. To conclude, SCC performs much better with less fines and well graded aggregate.

3.3 c) L – Box Tests Method:

1. Introduction:

The test assesses the flow of SCC and also the extent to which it is subjected to blocking by the reinforcement. As the diagram below shows the apparatus and its set up:

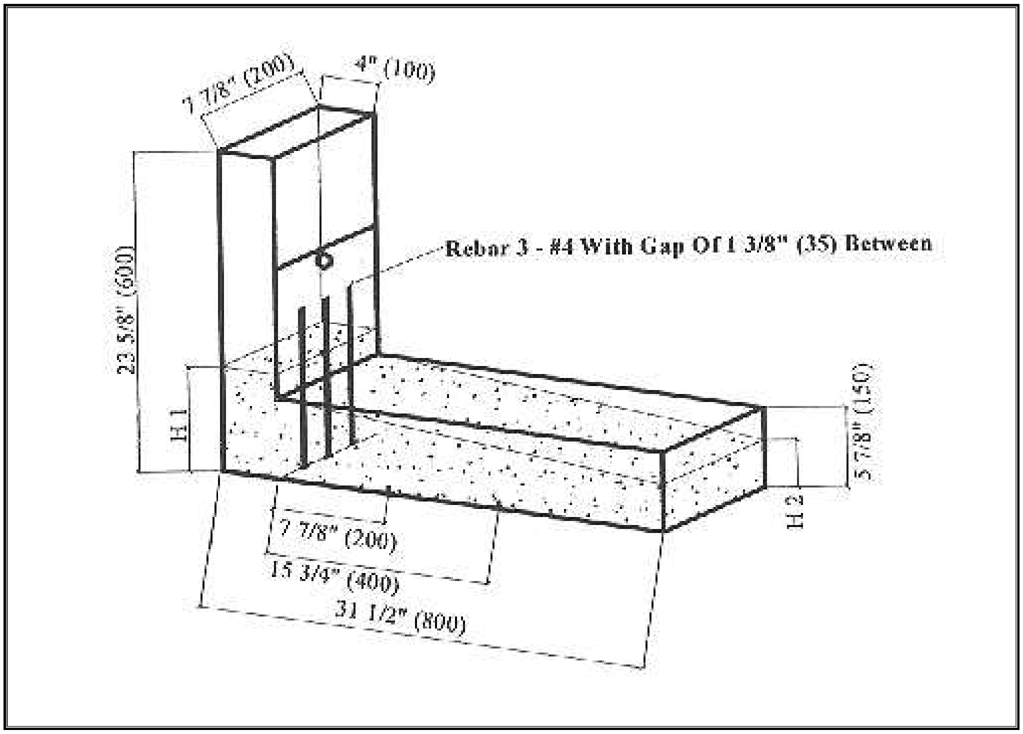


Figure 3.14: L-Box Test (PCI Guidelines)

The apparatus consists of a rectangular section box in the shape of an "L" with a vertical and horizontal section, separated by a moveable gate, in front of which vertical lengths of reinforcement bar are fitted. The vertical section is filled with SCC and then the gate lifted to let the SCC flow into the horizontal section. When the flow has stopped, the height of the SCC at the end of the horizontal section was expressed as a proportion of that remaining in the vertical section (H2/H1 in the diagram). It indicated the slope of SCC when at rest. This is an indication of the passing ability or the degree to which the passage of SCC through the bars is restricted.

However, the horizontal section of the box can be marked at 8 inches and at 16 inches from the gate and the times took to reach these points were measured. These are known as T-20 and T40 times were an indication for the filling ability.

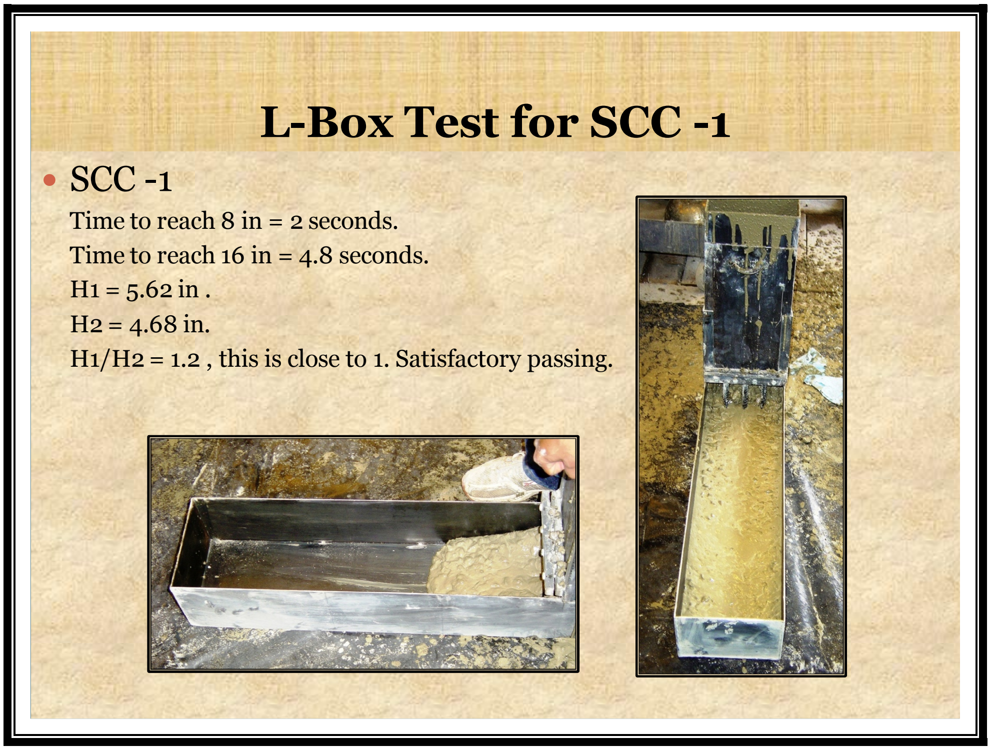


Figure 3.15: L- Box Test on SCC -1

2. Assessment of Test:

It assessed the filling and passing ability of SCC and serious lack of stability (segregation) could be detected visually. If the apparatus was designed for disassembly after the concrete was allowed to harden, segregation may also be detected by subsequently sawing and inspecting sections of the SCC in the horizontal section. However, this arrangement to some extent had the same replication as “the wall effect” that SCC might have on site when it is confined within the formwork.

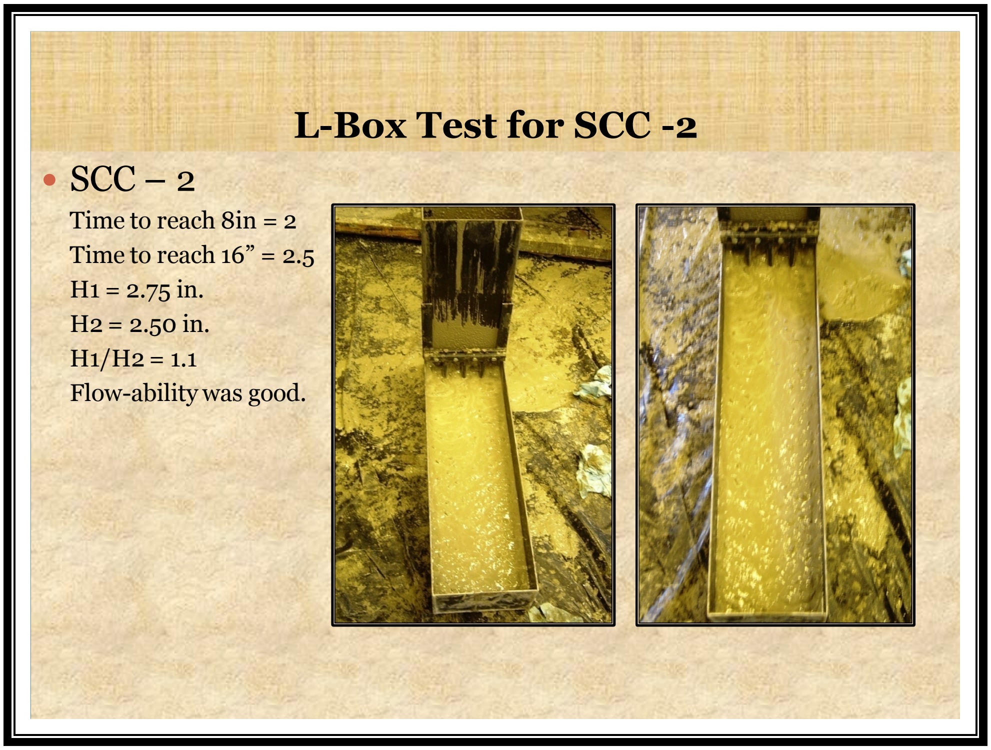


Figure 3.16: L-Box Test on SCC -2

3. Equipment:

a) L-Box of a stiff non-absorbing material.

b) A Trowel.

c) A Scoop.

d) A Stopwatch.

4. Procedure:

a) About 0.5 ft^3 of SCC was needed to perform the test, sampled normally.

b) The apparatus was set leveled on a firm ground, ensuring that the sliding gate can be opened freely and closed.

c) The inside surface of the apparatus was moistened and all the surplus water was removed.

d) The vertical section of the apparatus was filled with the SCC sample.

e) Then it was allowed to settle and stand for approximately for 1 min.

f) The sliding gate was lifted and SCC was allowed to flow out into the horizontal section.

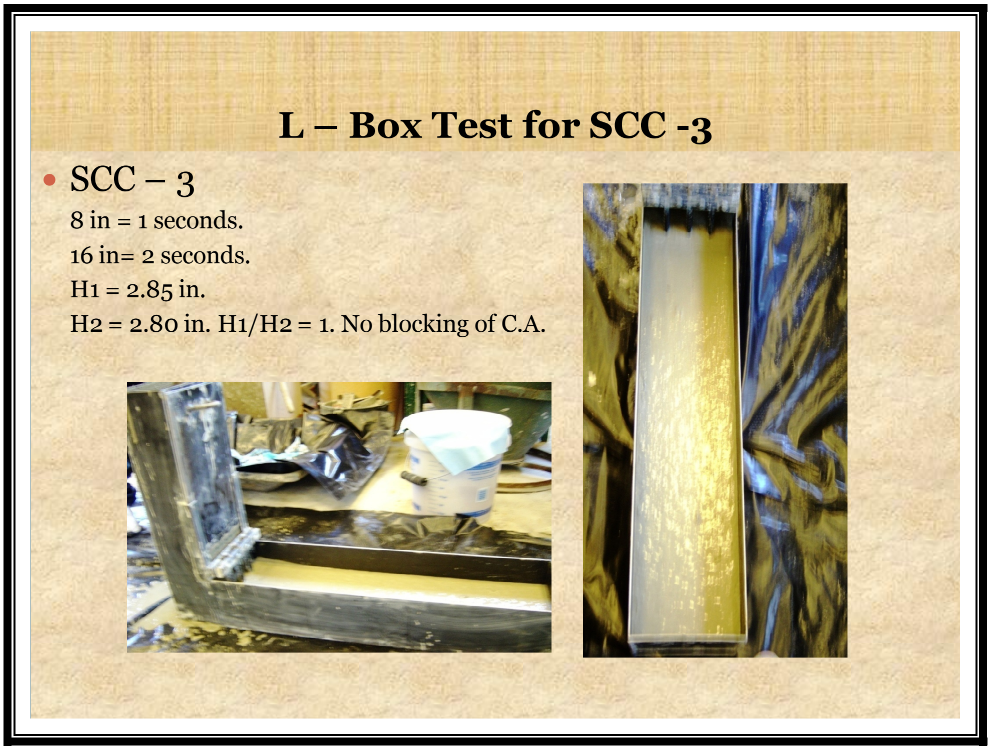


Figure 3.17: L-Box Test on SCC -3.

g) Simultaneously, the stop watch was started and the time taken for SCC to reach the 8in and 16 in marks were noted.

h) When SCC stopped flowing, the dimensions “H1” and “H2” were measured.

i) The blocking ratio H2/H1 was calculated.

j) The entire test was performed within 5 minutes.

5. Observation and Discussion:

For SCC-1 , SCC-2, SCC-3 the time taken to reach the 8in and 16 in marks in the L-Box were 2,4.8; 2,2.5; 1,2 seconds respectively. This shows that with reduction in the coarse aggregate size and content we will get enhanced filling ability, passing ability and flow-ability of SCC. Hence, with well gravels, and different kinds of finer particles like slag, fly-ash as cement replacement we can enhance the ability of SCC to behave much better than NC in various aspects. The ratio of H1/H2 showed that with lesser content and smaller aggregate size the blocking ratio reduced i.e; even in areas of congested reinforcements SCC can be used efficiently

6. Interpretation of the Results:

If the SCC flowed freely as water at rest, then it will be horizontal and H1/H2 = 1.The nearer this value is to 1.0 better is the flow of SCC. However, the blocking of the C.A behind the reinforcing bars was detected visually.

3.3 d) Comparison of SCC Mixes:

Slump Flow:

Figure 3.18: Slump Flow of

SCC-1 **SCC -2** **SCC-3**

The slump of all the mixes showed very less segregation and the VSI values were 0, 0.5 and 0 for SCC-1, SCC-2 and SCC-3 respectively. The addition of HRWRA and VMA are kept constant in order to keep the flow at a certain fixed range. As per the project specification, they can be altered in order to obtain a more or less slump. Also the T 20 times were 3.3 seconds, 2 seconds and 2 seconds respectively.

J-Ring tests:

The figure below describes, a comparison of the J-Ring test between different mixes were shown:

Figure 3.19 **SCC-1** **SCC -2** **SCC -3**

The J-Ring values were 0.429 in, 0.129 in and 0.625 in for SCC-1, SCC-2 and SCC-3. From the figure , it was clear that the blocking for SCC-1was more than SCC-2 and SCC-3. This means that, by using lesser coarse aggregate and more finer particles we can enhance the filling and passing ability of the mixes. This means that by increasing the fines we can increase its ability to pass through congested reinforcements.

L – Box Tests:

Figure 3.20: **SCC -1** **SCC -2** **SCC -3**

The L-Box ratios for the mixes are as 1.2, 1.1 and 1 for SCC-1, SCC-2 and SCC-3 respectively. The blockage behind the reinforcing bars was evident for SCC-1whose F.A to C.A was less than the other two mixes. It was found that the by increasing the fines to 10% in each single mixes the blocking can be reduced to 10% in each mixes. It can be concluded that by increasing the volume of the fine aggregates and the paste the filling ability can be enhanced. This is due to the fact that by continuous grading of the fillers (cement, fines) the inter-particle friction and collision was reduced to a greater extent and the cohesiveness of the mix can be enhanced by the better particle density.

3.4 Quality Control and Production Testing:

1. Aggregate:

The consistent production of SCC provided an added quality benefit to the architecturally exposed pre-cast products, for this close attention to adjusting the batch quantities for the free-surface moisture of both the F.A and C.A was important.

2. Concrete Strength:

ASTM C 31 is applicable to SCC except in the method of cylinder consolidation. Because of the fluidity of SCC, the cylinders was handled with extreme care and stored on a leveled stable surface until the cylinders were tested for stripping.

3. Slump Flow, T-20, VSI Tests:

The slump flow test, T 20 test and VSI visual rating of stability were the primary tools for measuring and evaluating the consistency of SCC production mixes between batches on a daily basis. Though, constant diligence was required for quality production, visual assessment of the mix, whether during a slump flow test or during the placement of SCC product is believed to be the eligibility criteria for acceptance of any SCC batch.

The T 20 test was performed independently of the slump flow and VSI test to provide the information on mixes that the test was applicable for.

The VSI rating gave us an idea about the qualification of the mix and its consistency. The mix design was adjusted as per need of the project specifications and the as per the designer.

4. Air Content:

The ASTM C 173 and C 231 are generally applicable to SCC.

5. Unit Weight:

The daily monitoring of the fresh concrete unit weight and yield is an excellent tool for discovering the trends in a concrete mix design. A change in the fresh unit weight indicated a change in the properties of the aggregates or an error in batch proportioning. ASTM C 138 is generally applicable to SCC as well only change is the method of consolidation.

6. Temperature and Air Temperature:

The temperature of the SCC and the surrounding temperature were kept in track with every batch in order to avoid any huge deviation from the normal circumstances. Thus, when concrete was made in extreme climates like hot weather or cold weather conditions extra precautions are to be taken as compared to the normal concrete.

3.5 SCC Concrete Properties:

As mentioned above, all the concrete properties were evaluated as per the mentioned ASTMs and their required quality control. The following table showed all the properties as follow:

MIXTURE DESIGN PROPERTIES:	C-N	SCC-1	SCC - 2	SCC -3
Fresh Concrete Properties				
Slump flow spread (in)	8	25.15	27	26.15
Inverted Slump Flow (in)	-	23.7	24.6	23.9
T 20 seconds	-	3.3	2	2
J- ring (in)	-	0.43	0.13	0.62
L- Box Filling Head Drop (H1/H2) in		1.2	1.1	1.0
Air Content (% by volume)	1.5	0.5	0.2	0.8
Unit weight (pcy)	149	148	146	147
Temperature ©	24	30	25	25
Water/Cement Ratios (w/c)	0.46	0.47	0.47	0.45
Hardened Concrete Properties				
Compressive Strength (Type 3)				
1 day (psi)	4994	4615	4443	4866
7 day (psi)	6844	6140	6048	6697
28 day (psi)	7792	7083	6685	8256

Table: 3.3 Summary of Fresh and Hardened Properties of SCC Mixes

Note: The explanation for the SCC mix designs and their fresh properties were already explained earlier.

3.6 Discussion of Test Results:

The table below summarizes the effect of the increasing the F.A to C.A ratios in the SCC mixes. All these mixes were designed to evaluate the effect of SCC on pre-stress bond, which is discussed in the next chapters.

	NC	SCC-1	SCC -2	SCC -3
F.A/C.A	0.665	0.713	0.816	0.902
Slump	8	25.15	27	26.15
J-Ring		0.43	0.13	0.62
L-Box		1.2	1.1	1
T8		2	2	1
T16		4.8	2.5	2
T20		3.3	2	2

Table 3.3: Summary of SCC Design Mixes

It is evident from the table that as compared to the normal concrete the ratio of fine to coarse aggregate was much higher in SCC mixes. The fresh properties as discussed earlier was effect of super-plasticizers the HRWRA and the VMAs. The 10% increase in the fines in all the mixes had a increased effect in the filling and passing ability of the SCC as found by the J-Ring and L-Box values. The flow-ability of the mixes were found from the T8, T16 and T20 timings. There is greater enhancement in the flowing time , which means that the SCC is capable of filling more easily horizontally with and without any blockages in the form of congested reinforcements. To conclude, it can be said that with the higher paste volume, the voids between the aggregates can be filled easily and the layer enveloping the aggregate particles helps in achieving higher deformability and segregation resistance. It can also be concluded that viscosity of the paste is directly proportional to the average aggregate spacing and shape (size of coarse aggregate). To add , the optimum flow viscosity can be achieved by suitably designing the mix and controlling higher aggregate spacing and aggregate diameter (size of coarse aggregate).

CHAPTER IV

NASP PULL OUT TEST

4.1 Scope of Research:

The objective of the research was to determine whether the NASP Pull out test was suitable for adoption as the Standard Test for Strand Bond to assess the ability of the pre-stressing strand to bond with concrete. In order to start with the NASP bond test, a brief literature review of how the various bond tests that were

4.2 Pull Out Tests:

Cousins, Badeaux, Moustafa (1992)

The experimental research (Cousins 1992) aimed at the comparative study between a test methodology introduced, and a direct tension pull out test. The research was to develop a standard test for determining the bonding characteristics of epoxy coated and uncoated pre-stressing strands to concrete and to correlate them to the transfer lengths. Low relaxation Grade 270 pre-stressing strands with both uncoated and epoxy coated grit impregnated strands were used for the test program. After pre-tensioning the strands to the desired levels, a concrete block was casted around the strands. The concrete block was forced out of the strands and the values recorded using a hydraulic actuator. The load vs. strand slip was recorded with respect to the concrete to determine the failure force. Linear Variable Differential Transducers were used to monitor the load. It was concluded from this research that the standard test gives higher bond stress at initial strand slip than the direct tension pull out test. Transfer length made with 3/8 in diameter uncoated strand resulted in a smaller bond stress than that from the standard results.

The authors also concluded that the grit density variations and the rusting of strands resulted in higher standard deviations. The research did not provide significant relationship of the test with the transfer lengths.

Logan (1997):

A study was conducted to understand the bond behavior of pre-stressing strands with concrete in pre-stressed concrete applications. In order to conduct the research, six 0.5 in. diameter strands were used.

To conduct the Moustafa pull out tests, strand specimens which are 34 in. long were saw-cut, towel wiped, and straightened for bow before casting. The strands were embedded in a block with light reinforcement and concrete with 4000 psi one day strength and 28 day strength of 6000 psi. The concrete test beams were single strand sudden pre-stress release using flame cutting was performed on all the test beams. Logan concluded from this research that Moustafa test is a reliable test to predict the flexural behavior of beams. The strands having pull-out capacities less than 12 kips equaled the ACI transfer length equation at release however, the transfer length increased over time. . The research also concluded that the color of the strand, surface residue from the wipe test, and the lay or pitch of the strands did not reflect on the bond potential of the strands. Some of the "as received" strands outperformed the strands which had light rust on the surface. Logan recommended that the 0.5 in. diameter strands used in the industry require a pull-out capacity of 36 kips with a 10% coefficient of variation for a sample set of six specimens. Moustafa test should be performed for repeatability with different concrete mixtures.

Ferzli, Y.(2000) :

The research program inspects the Moustafa Pull out test in assessing the bond performance of pre-stressing strand with concrete (Ferzli 2000). The research variables included strands from five different manufacturers for pull out tests and for flexural tests consisting of single and double strand beams. A total of 24 rectangular beams and 72 pull out tests were conducted.

The study concluded that the Moustafa Pull out capacity was inversely proportional to the transfer length. However, results did not show strong correlations between theMoustafa Test and transfer length. Ferzli concluded that the Moustafa Test was unacceptable as a test method to assess the bond performance of pre-stressing strand with concrete. Regardless, the author recommends Moustafa test as a preliminary test procedure to assess the general bond qualities of pre-stressing strands. The experimental work showed that the strand slip increases over time and can grow upto 33% of its release value. The author recommended to perform a more reliable test method with fewer variables than the Moustafa Test.

NASP Round 2 (1999)

The North American Strand Producer's (NASP) funded two research projects (NASP Round I, II) to evolve a standardized test procedure to understand the bond characteristics of prestressing strands. The program investigated various pull out tests including Moustafa Pull out test, Post Tensioning Institute (PTI) Bond test, and friction bond pull out test, (Paulsgrove 1999) which were conducted at University of Oklahoma and Florida Wire & Cable Inc. During the NASP Round I series, a friction bond test was conducted after mechanically splicing the strands together. Strands were mechanically spliced by a mechanical coupling. The test measured the force required to pull the splice apart. The spliced strands were placed in a hydraulic device exerted a uniaxial tension. These results did not show convincing degree of reproducibility. The PTI Bond test measured the pull out value for 0.10 in. (2.54 mm) slip using the procedures by the Post Tensioning Institute (PTI). The PTI specimens were cast in 5in. diameter and 18 in. tall steel casings with a strand located concentrically in the specimen. The steel casings were welded to a base plate 6 in. by 6 in. by 0.25 in. thickness. A 9/16 diameter hole accommodated the strand to pass through the mould.

The strand had a total length of 40 in. with an effective embedment length of sixteen in. The mould was filled with “neat cement grout” after placing the strands in position. Due to high shrinkages, 0.5 in. in a specimen, the specimens had to be flushed with mortar after 30 minutes of pouring the moulds. When the grout attains strength in the range of 3500 to 4000 psi the PTI Bond test is conducted. A displacement controlled test method is employed with a rate of loading of 0.1 in. per minute. The free end slip is digitally recorded using an LVDT attached on the strand and measured relative to the top of the flushed mortar.

The NASP test employed similar procedures as the PTI bond test, except for a few variations. Sand was added to the mortar mix to reduce the amount of shrinkage and provide more consistency within the mix which also included Type III cement and water. Apart from slight variations in placement and the vibration techniques, the test methodology was similar.

The NASP Round II test program compared the data from both the test sites for Moustafa, PTI and NASP Bond test. The results from the data are presented to understand the reproducibility of the test methods in Figs. 2.8 through 2.10. The results from OU and FWC are compared for Moustafa, PTI, and NASP Bond test. Figure 2.11

showed the coefficient of regression of all the test methods during NASP Round II. The PTI and the Moustafa bond test reported the maximum force at 0.1 in free end slip, whereas, the NASP test reports the data for 0.1 in. free end slip as they showed the least variation in the data. Recommendations were made to further conduct research on NASP test.

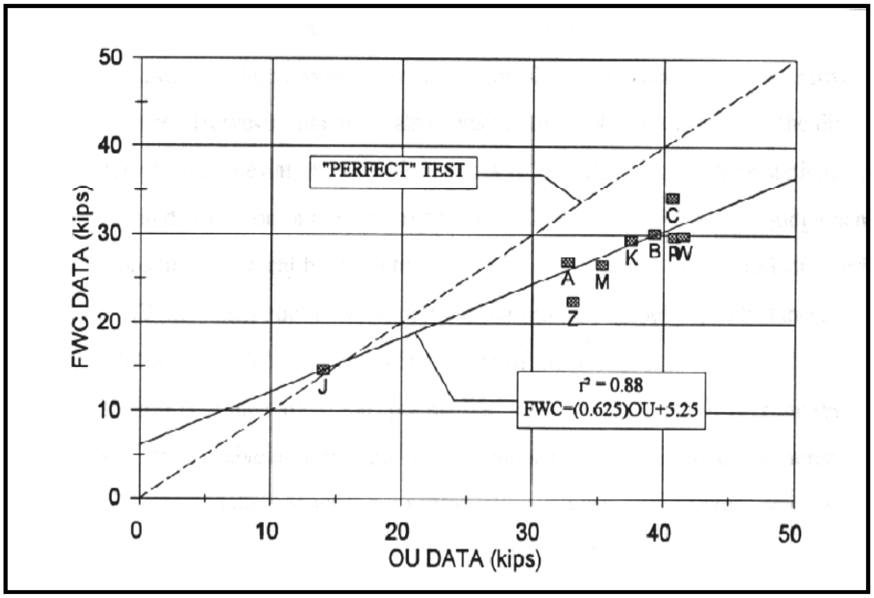


Figure 4.1 : Moustafa Test Results from OU and FWC (Paulsgrove)

TABLE 3.2
PTI BOND TEST RESULTS
MAXIMUM PULL-OUT CAPACITIES
NASP ROUND TWO TESTING

	OU TEST DATA (sample size = 6; 5 for K)			FWC TEST DATA (sample size = 6)		
STRAND	PULL-OUT (kips)	Std. Dev. (kips)	C.V. (%)	PULL-OUT (kips)	Std. Dev. (kips)	C.V. (%)
A	12.61	0.84	6.7	14.19	0.84	5.9
B	11.70	0.89	7.6	14.88	1.29	8.7
C	13.41	1.22	9.1	15.34	2.00	13.1
J	5.08	0.93	18.4	5.98	0.85	14.1
K	12.62	1.25	9.9	14.49	1.84	12.7
M	12.03	1.38	11.4	14.13	1.02	7.2
P	13.83	1.03	7.5	16.60	1.79	10.8
W	11.01	1.17	10.6	12.70	2.21	17.4
Z	9.80	0.82	8.4	7.95	0.83	10.4
AVERAGE	11.34	1.06	9.96	12.92	1.41	11.14

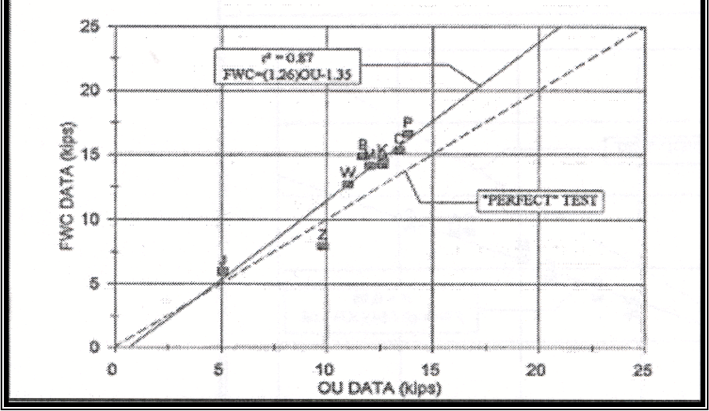


Figure 4.2: PTI Test Results from OU and FWC (Paulsgrove, 1999)

TABLE 3.6
NASP ROUND TWO TESTING
NASP BOND TEST RESULTS - SERIES ONE
PULL-OUT FORCE AT 0.10 in. SLIP

	OU TEST DATA (sample size = 6)			FWC TEST DATA (sample size = 6)		
STRAND	PULL-OUT (kips)	Std. Dev. (kips)	C.V. (%)	PULL-OUT (kips)	Std. Dev. (kips)	C.V. (%)
A	17.70	2.09	11.8	12.48	3.42	27.4
B	11.81	1.20	10.2	8.02	2.69	33.6
C	19.57	1.96	10.0	12.85	2.64	20.6
J	2.61	0.57	21.7	2.77	0.64	23.2
K	13.76	1.71	12.4	9.32	2.79	29.9
M	14.87	2.01	13.5	10.69	2.49	23.3
P	17.12	1.65	9.6	12.47	1.77	14.2
W	10.35	1.54	14.9	6.77	1.67	24.7
Z	5.68	1.19	21.0	5.17	1.36	26.2

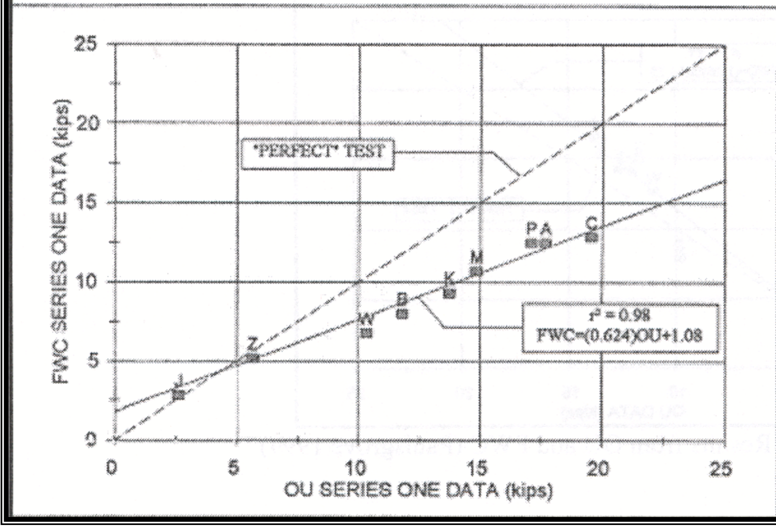


Figure 4.3 : NASP Test Results from OU and FWC ,(Paulsgrove 1999)

TABLE 3.11
NASP TESTING ROUND TWO
SUMMARY OF REGRESSION ANALYSIS

Data Comparison	Moustafa	PTI Pull-Out			NASP Pull-Out		
	Maximum Strand Force	Maximum Strand Force	Force at 0.10 in. SLIP	Force at 0.01 in. SLIP	Maximum Strand Force	Force at 0.10 in. SLIP	Force at 0.01 in. SLIP
	r^2	r^2	r^2	r^2	r^2	r^2	r^2
Series 1 vs. Series 1							
OU vs. FWC	0.88	0.87	0.90	0.73	0.97	0.98	0.90
OU vs. STRESSCON	0.92						
STRESSCON vs. FWC	0.94						
OU Series 1 vs. OU Series 2					0.97	0.97	0.71
OU Series 1 vs. FWC Series 2					0.87	0.97	0.72

r^2 (coefficient of determination)
regression about "best fit" line

Figure 4.4: Summary of Regression Analysis (Paulsgrove,1999)

Brown and Russell (2003):

Studies were conducted as a part of the NASP Round III at Oklahoma University (OU) and Florida Wire & Cable (FWC) to investigate the NASP, PTI and Moustafa bond tests (Brown 2003). Ten samples of strands were tested at OU and FWC to investigate the reproducibility of all the test procedures. Moustafa tests performed as a part of this research conformed to Logan (Logan1997) test methodology.

Maximum Pull Out Values for Moustafa Test (kips) Sample Size = 6								
Strand ID	**Max.Pull Out Value**		**Standard Deviation**		**Coeff. Of Variation**		**f'c**	
	OU	**FWC**	**OU**	**FWC**	**OU**	**FWC**	**OU**	**FWC**
AA	33.04	26.6	1.052	5.612	3.2	21.1	4220	4270
BB	29.47	28.24	2.27	1.524	7.7	5.4	4220	4270
CC	31.12	22.18	2.41	1.024	7.7	4.6	4220	4270
DD	36.45	27.17	3.84	1.709	10.5	6.3	4970	4270
EE	37.44	25.97	2.283	1.767	6.1	6.8	4970	4270
FF	22.6	21.02	1.059	1.899	4.7	9	4970	4270
GG	26.33	26.2	3.646	0.729	13.8	2.8	3580	4270
HH	35.05	25.14	2.609	1.397	7.4	5.6	3580	4270
II	18.91	13.43	2.214	1.746	11.7	13	4340	4270
JJ	25.07	31.93	2.424	2.918	9.7	9.1	3580	4270

All the 10 strand specimens were tested for Moustafa pull out test. Each Moustafa block contained 18 strands and a total of 60 strands were tested in all. These tests were performed both at OU as well as at FWC. The results obtained were tabulated in the previous page.

The PTI Bond test procedure was similar to the NASP Round I, Round II methodology as explained earlier. The neat cement grout was poured into the specimen cylinder and the test was conducted at a loading rate of 0.1 in. per minute. The tests were conducted at University of Oklahoma and Florida Wire & Cable to check for the repeatability of the test method. The results from the PTI Bond test are presented in Table 4.2 below:

Average PTI Pull Out Values (kips) at 0.1" Slip									
Strand ID	NASP Pull Out Value		Sample Size of test		Standard Deviation		Coeff. Of Variation		f'c
	OU	FWC	OU	FWC	OU	FWC	OU	FWC	OU
AA	9.64	11.23	6	6	0.58	1.9	6	16.9	3450
BB	6.68	7.13	6	6	0.64	1.51	9.6	21.2	3470
CC	5.62	7.93	6	6	0.86	2.23	15.3	28.1	3440
DD	6.14	11.01	6	6	1.21	2.7	19.7	24.6	3540
EE	6.94	10.12	6	6	0.53	2.25	7.7	22.2	3560
FF	4.58	7.34	6	6	0.55	2.89	11.9	39.4	3450
GG	7.21	7.34	6	6	0.38	1.13	5.2	15.4	3650
HH	9.01	8.61	6	6	1.26	1.62	14	18.9	3670
II	5.5	5	6	6	0.45	2.92	8.1	58.5	3510
JJ	7.41	10.67	6	6	0.7	4	9.6	67.5	3490

Table 4.2: Average Pull Out Values at 0.1" Slip.

The NASP Pull out tests were conducted at OU and FWC on all the strand sources. The test was conducted in a sand cement mortar using a displacement controlled testing at a rate of 0.1 in. per minute. The NASP test procedure was not modified from the NASP Round II procedures. Table 2.4 reports the average of NASP pull out value at 0.1 in. of free end slip on a sample size of six specimens at OU and FWC. The results from the research program concluded that the Moustafa pull out test was not a reliable test for bond. A coefficient of determination of 0.0476 was seen when plotting the data between OU and FWC. However, the test was able to relatively predict the bond performance ranking.

The PTI bond test was not too different as far as reliability to produce reproducible results is concerned. The correlation of determination for the PTI bond test was reported as 0.2104 when the data was plotted between OU and FWC. The problem of shrinkage was considered as the cause of inconsistency in the test procedure. The NASP test "outperformed" the other bond tests, showing a strong correlation between both locations having an R_2 value of 0.776. The test proved as a reliable predictor for bond when compared to other testing procedures. The researchers recommended that the NASP test should be investigated further to make stronger conclusions.

Average NASP Pull Out Values (kips) at 0.1" Slip										
Strand ID	**NASP Pull Out Value**		**Sample Size of test**		**Standard Deviation**		**Coeff. Of Variation**		**f'c**
	OU	**FWC**	**OU**	**FWC**	**OU**	**FWC**	**OU**	**FWC**	**OU**
AA	13.93	15.97	6	6	1.248	2.694	9	16.9	4370
BB	6.75	10.37	6	6	0.719	0.963	10.6	9.3	3810
CC	9.93	8.8	6	4	2.506	1.379	25.2	15.7	3970
DD	14.35	15.26	6	6	0.598	1.754	4.2	11.5	4150
EE	14.09	16.02	6	6	0.587	4.168	4.2	26	4320
FF	6.31	8.29	6	6	0.409	1.291	6.5	15.6	3900
GG	7.17	12.41	6	6	1.004	1.259	14	10.1	3730
HH	11.12	10.29	6	6	1.002	1.639	9	15.9	4100
II	2.98	5.3	6	6	0.319	0.843	10.7	15.9	4000
JJ	19.68	17.61	6	6	1.401	3.153	7.1	17.9	4220

Table 4.3: Average NASP Pull Out values at 0.1"Slip

According to Chandran (2004), the NASP test showed the highest correlation between the pull out value and the transfer lengths measured at release and at 28 days. A correlation of determination of 0.86 was seen on the strands when the NASP values and the transfer lengths at release were plotted for the single and double strand beams. The correlation of determination for

Moustafa pull out test was reported as 0.61 and for PTI bond test was 0.70. This further proved that the NASP bond test is a better predictor for bond than Moustafa and the PTI bond test.

Summary:

Thus, in order to substantiate the efforts of correlating the transfer lengths with varying concrete strengths numerous research were performed by various researchers. Among these tests, NASP pull out test showed convincing results. However, a standardized acceptance procedure to quantify the pre-stressing strands is unavailable in spite of the scatter in the transfer length data.

4.3 Material Properties:

4.3.1 Pre-stressing Strands:

As mentioned earlier, the pre-stressing strands used for this research program were seven wire 270 ksi Low relaxation strands from manufacturers in North America. The strands conformed to ASTM 416 specifications for Low relaxation strands as attached in Appendix .The pre-stressing strands had a nominal diameter of 0.5 in (12.7 mm). The nominal cross-sectional area was 0.153 sq in (98.7 mm^2) for 0.5 in diameter strands. The modulus of elasticity of the pre-stressing strands was estimated as 28,500 ksi (196.3 Gpa).

4.3.2 Mortar:

The mortar used for the NASP testing had a specific strength in the range of 4750 to 5000 psi. All the sand and cement were mixed in different proportions and trial batched in order to meet that certain strength. The specifications of the sand and the cement are attached in the Appendix at the end.

4.3.3 Sample Preparation:

The sample size was exactly 32’’. After the sample was cut, it will be subject to grinding and thus the cut sample should always be between 32 1/8’’ and 32 1/16’’. A sample smaller than 32’’ will not conform to test specifications and a sample much larger than 32’’ will cause difficulty during the testing procedure.

The sample had two ends the first end can be cut at the place that has been marked. In order to cut the other end the sample should be re-measured to 32’’ and then cut. By this any error in measurement was avoided.

The marked portion of the strand that needs to be cut was put on the chop saw. The marking was aligned with the blade of the saw such that the edge of the cutting blade will lower exactly on the marking. The strand was then tightened in position on the vice. Then the position of the blade relative to the marking was checked again. The saw was then turned on and blade brought to the maximum speed. After the cutting blade had reached its maximum speed it is then lowered on to the strand.

Sample numbering

The strand sample has duck tape on both ends.

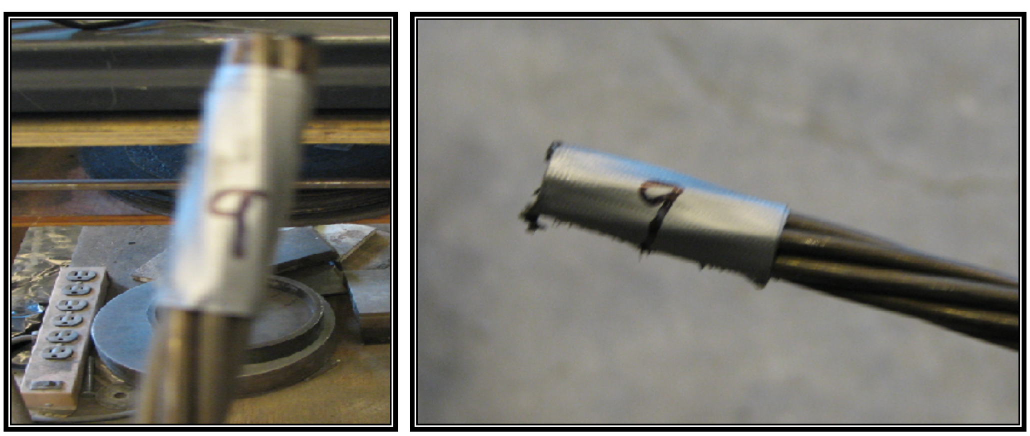


Figure 4.6: Plant Number and Sample Number Marked.

Grinding Samples:

The ends of the strand sample were ground on a belt/disc grinder as shown in Figure. Ensure that the sanding belt and sanding disc are not worn-out. If so, change them. The samples of a particular plant to be ground should be carried to the grinding machine in a box appropriately labeled. This will ensure that the samples are not misplaced or contaminated. Both sides of the strand sample need to be ground, to different extents. The strand sample is first examined by looking to check if it was excessively bowed and to find the position of the bow on the strand. If the sample is excessively bowed then we cannot use it. If an acceptable bow lies in the middle of the strand, both ends can interchangeably ground. If the bow exists towards one end of the strand, we look for the end that gives us the longest length before an obvious bow. That end should be ground at 40 degrees to approximately 5/8 in. and the end close to the bow should be ground at 50 degrees to approximately 1/8in.

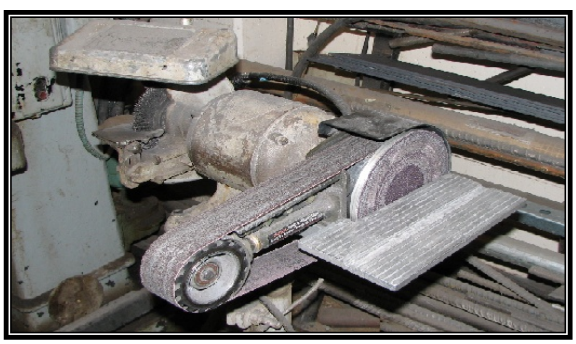


Figure 4.7: Belt and Disc Grinder.

First flatten the cut ends of the strand sample using the disc grinder as shown below:

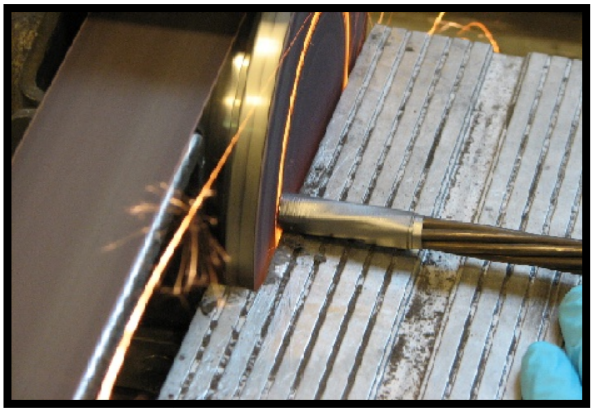


Figure 4.8: Flattening Ends of Sample

After this, using the disc grinder, grind both ends of the sample at approximately 50 degrees angle for an approximate length of 1/8 in. as shown in Figure 3.8. These two steps will ensure that the strand will be in proper shape and no unwanted metal will be on the ends of the sample.

Figure 4.9: Ground End at angles.

It will also ensure that the pre-stressing chuck fits properly onto the strand and will not be obstructed by melted strand ends.

Now choose the end that needs to be ground at 40 degrees to 5/8in. as explained in the general requirements and use the belt to sharpen it as shown in Figure**.** While sharpening the edge ensure that the middle wire of the strand, i.e. the king wire, is exposed as shown in Figure**.** The outer strands in fig are darkened to show the exposed king wire. The exposing of the king wire while grinding will enable the measuring devices to measure the centerline slip during testing.

Figure 4.10 (a): 5/8in. Ground Side

Figure 4.10(b): Exposed King Wire

At the grind the strand should not appear burnt or the king wire should not be ground to a pencil point. If the strand is ground to a pencil point, leveling the Linear Variable Displacement Transducer (LVDT) support assembly on it will be very difficult.

The essence of grinding and exposing the king wire is that at the time of pre-stress transfer as discussed earlier, the strands slip in the concrete. Though , due to the helical nature of the strands and mechanical interlocking the outside wires get restrained, as they failed to twist, whereas the king wire slips. This slip was found to be 0.1in as explained earlier and proved from research by researchers. The king wire was exposed in order to determine the slip appropriately by the external LVDT.

Strand Sample Bond Breaking:

Process:

First lay out the strand to be bond broken on the table. Using the guide and a marker mark the point on the strand above which the bond breaker will sit as shown in Figure. Set the marked strand on the table. Take the 1/2in. styrofoam pipe insulation and cut a length of 2in. Slit open the two inch piece with the help of scissors. Now try to wrap the slit styrofoam piece over the strand. The strand will float in the piece. Trim the styrofoam piece such that it wraps tightly around the strand as shown in Figure . Now cut two piece of brown packing tape one of length approximately 4 in. and the other of approximately 3in and set them on the edge of the table. With one hand, hold the styrofoam piece tightly around the stand above the marking and with the other hand rap the 4in. piece of tape around it**.** After doing so check the sample with the guide to make sure the bond breaker fits firmly on the guide as shown in Figure.

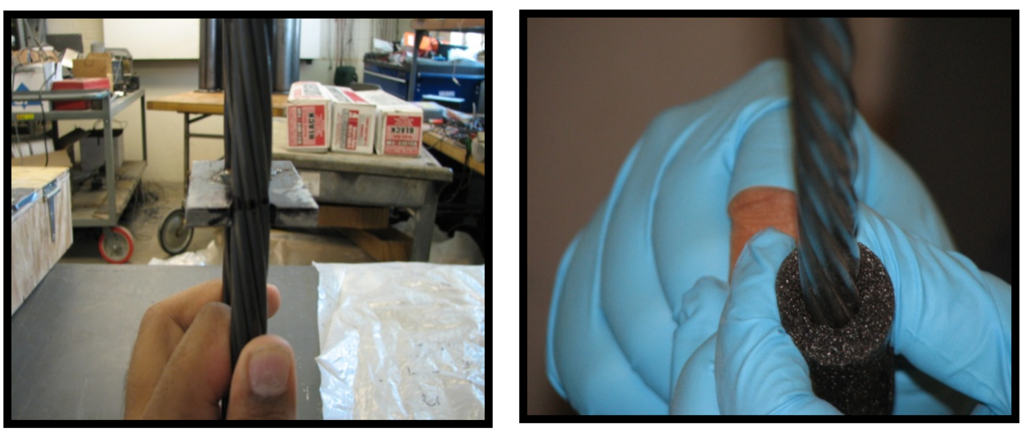


Figure 4.11: Attaching the bond breakers on the strands.

Significance of Bond Breakers:

The importance of bond breakers lies in the fact that there will be no abrupt stress concentration at the joint of the strand and the base plate during the pull out tests.

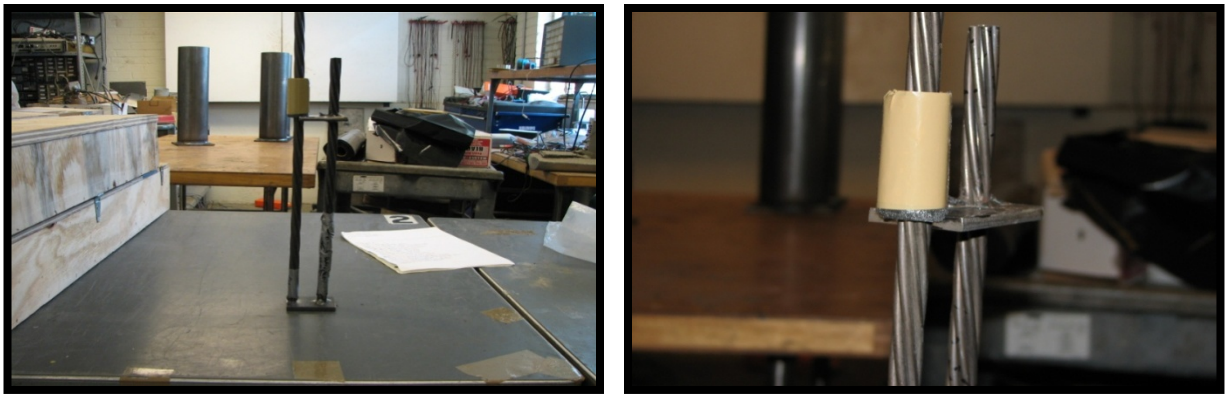


Figure 4.12 : Making Sure the Bond Breaker is at the Right Level.

Then hold the strand sample at a point above the bond breaker with one hand such that the bond breaker stays in position and tape the lower end with the 3 inch piece of tape with the other hand. If in doubt whether the bond breaker has shifted position, use guide to check position. If the bond breaker is above the required level, there will be loss of water from the mortar during casting. If the bond breaker is below the required level, the strand will dangle in the form; also the chuck may not be able to grip the stand during testing. A correctly bond broken sample is shown in Figure 4.13.

Figure 4.13: Bond Broken Strand Sample.

Cylinder Assembly:

Cylinder assemblies are basically molds in which the strand samples are cast. A cylinder assembly is made up of 11gauge or thicker tubing, 5in. external diameter and 18in. length welded to a 6in X 6in X ¼in plate. The plate has a 9/16 in. hole at it's center. Plate and the cylinder are to be welded such that the center of the plate hole is in line with the center line of the cylinder.

We also call the cut 6in length of strap steel as plate and 18in. length of tubing as cylinder. The plate and cylinder on tacking and complete welding is called a cylinder assembly. The lengths of samples being cut should be checked to ensure they are 18 1/16'' each.

4.3.4 GRINDING CUT CYLINDERS

The cut cylinder has two rough cut faces. One face sits on a plate to be welded and the other face is exposed and is sometimes used to grip the cylinder assembly while moving it. Both the cut faces need to be ground. Grinding the face to be welded will ensure that the cylinder sits flush with the plate and grinding the exposed end will provide a smooth edge to grip while moving the cylinder assembly. The process of grinding can be performed in many different positions. One way to do it is to lay the cylinder along its length and grip it with one hand. With the grinder in the other hand move the grinder along the cut edge of the cylinder to grind it.

Process

The length of the plate is 6''. A stop block should be clamped at a distance to 6 1/16'' form the face of the band as shown in Figure 4.14 . The 1/16'' extra will be ground of during the grinding process and will ensure that we have a 6 inch plate. Once the stop block is clamped in position the band saw should be raised, locked in position and the 6'' plate to be cut, brought in position. While putting the plate in position we need to make sure it slides in perpendicular to the blade and the stop block such that the cutting face of the plate is parallel to the block. After the tube is in position, it should be secured to the saw using the vice.

Figure 4.14© : Measuring To Cut Plate.

4.3.5 BOTTOM PLATE MARKING AND DRILLING

Process

Lay a plate on the table and using a punch and scale, mark the diagonals of the plate as shown in Figure 4.7. Make sure to see that the diagonals run from corner to corner. Using the punch and a hammer, punch the point of intersection of the diagonals as shown in Figure 4.8. This point should be the exact centre of the plate. After the plate is punched it is ready to drill. The plate should then be fitted on the drilling vice and a 9/16’’ drilling bit adjusted such that the tip of the drilling bit coincides with the center of the punched hole on the plate. After adjustments are made, tapping oil is sprayed on the punched part of the plate. The drill press is switched on low and the drill bit is lowered to meet the plate. As the drilling of the hole is in progress, tapping oil should be spayed onto the drill bit to keep it from overheating and burning off.

Figure 4.14(a) : Diagonals of the Plate Marked Figure 4.14(b):Point of Intersection marked.

This oil also lubricates the drilling process. The hole should be drilled through the plate and it should be made sure that the bit slides comfortably in and out of the hole as shown in Figure .

Figure 4.15 : Plate Drilled Right Through

4.3.6 BOTTOM PLATE GRINDING

Process

There are four areas on the drilled plate that need to be ground. The band saw cut edges and the portions around the drilled hole on both sides of the plate. The order in which they are ground is not important. Make sure that the area around the drilled hole is ground properly. If not ground properly flakes of metal will project from the drilled hole and the plate will not fit properly onto the jig while tacking to the cylinder. Even if it does fit properly on the jig the flakes of metal if on the inside of the can will cause mortar water leakage while casting.

Tack Welding of Cylinder Assembly:

Tacking was carried out on the jig as shown in Figure 4.10. The jig was so made that the centerline of the hole in the plate coincides with the center line of cylinder. At first

Figure 4.16 : Jig for Tacking Cylinder to Plate

the plate is mounted on the jig such that the nut on the jig goes through the hole in the plate. The edges of the plate are kept parallel to the base of the jig. Then the cylinder is mounted on the plate and fastened to the vertical edge of the jig with a C-clamp as shown.

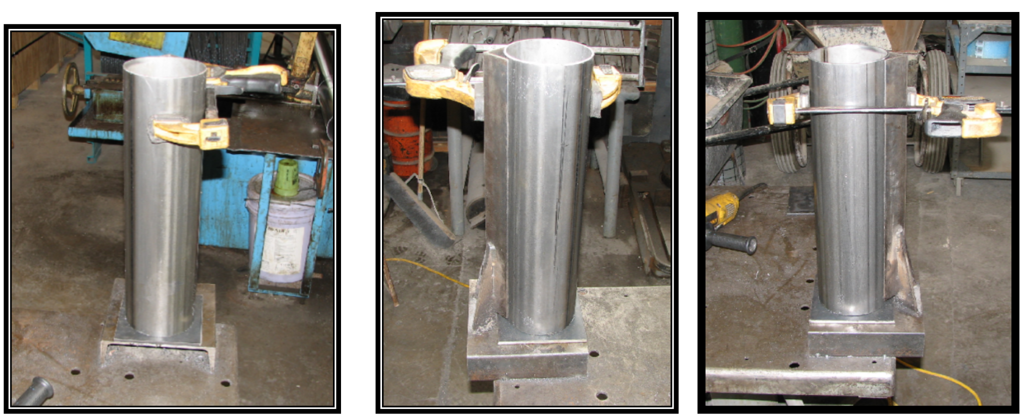


Figure 4.17: Cylinder and Plate on Jig and Secured by Clamp.

At this point it was important to check if the cylinder was properly clamped to the jig. If the cylinder was not clamped in position properly it will cause eccentricity between the centerline of the plate hole and that of the cylinder. If this occurs the cylinder cannot be used in the test. With the plate and cylinder clamped on the jig and the welder properly grounded, the cylinder is tacked to the plate. The cylinder is usually tacked at four points to ensure that the tack holds the plate and cylinder together.

4.3.7 WELDING OF CYLINDER ASSEMBLY

Guidelines

The aim of welding the cylinder and plate together is to ensure water tightness.

Figure 4.18 : Completed Cylinder Assembly

PRE-BATCHING PROCEDURES:

The casting of test specimens called “batching” is carried out after the strand samples and cylinder assemblies are prepared. However, after the strand samples and cylinder assemblies are ready and before batching, some time is spent setting up to batch. This setting up to batch consumes anywhere between 30-45 minutes, depending on the number of specimens to be cast and the people setting up.

At this stage we assume that the strand samples are prepared and cylinder assemblies are ready.

CLEANING 2in. CUBE MOLDS

For every batch of twelve samples, four molds of 2in. cubes are cast, each mold contains three cubes. The molds should be cleaned with a wire brush and oiled with form oil to make sure that the mortar does not stick to the sides of the molds. Form oil should be applied just sufficient to cover all surfaces of the mold that can be exposed to mortar. Make sure that all inside edges of the mould are brushed with the oil. A set of oiled cubes is shown in Figure.

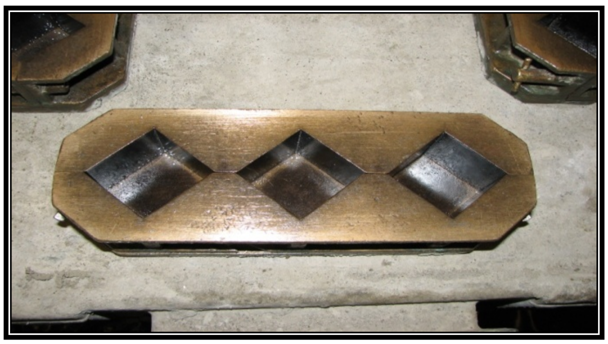


Figure 4.19: Oiled Cube Molds

4.3.8 FLOW TABLE SETUP

The flow mold and table should be cleaned and the table lubricated with the appropriate oil. The drops of the flow table should be checked with the mould on the table and turning of the crank handle. The flow table set up should be accompanied by the measuring caliper and rubber tamper as shown.

Figure 4.20: Measuring Mortar Flow in a Flow Table.

4.3.9 MIXING PROCESS

The mixer used in the lab is a shear mixer. This mixer has a counter-current mixing action. Due to this counter-current action the pan rotates in one direction, while the mixing tool rotates in the other. The rotating mixing pan conveys the material to the counter-rotating tool assembly also called “paddles”. This results in a uniform distribution of all ingredients in the finished batch.

In order to clean the mixer, its mixer paddles were raised above the level of the pan using the manual head lift assembly. The pan was lifted from the base frame and mixing pan assembly and kept on it side on the floor. Using a wire brush the paddles were cleaned to ensure that no sand particles or hardened mortar from previous batches mixes with the current batch. The pan was then checked to make sure it is clean. If not clean, it should be brushed with a wire brush. It was then brushed with form oil. Excess application of form oil was avoided.

SETTING UP FORMS WITH CANS AND STRAND SAMPLES

A sheet of plastic should be spread at the batching area and wooden forms set over it. The forms should be set at a clear distance from the concrete mixer as shown in Figure below. This will ensure that the mixer pan can be taken out from the mixer without hitting the form. The cylinder assemblies should now be placed on the forms.

Figure 4.21(a): Forms at a Clear Distance from Mixer

Figure 4.21(b):Numbering of Cylinder from Assembly

Once the cylinder assemblies are numbered the strand samples can be put in them. The strand samples should be carried out to the batching site in their respective boxes and should not be brought out individually. If two different plants are being tested, strand samples from one plant should be brought out at a time. It is important to check that sample plant number and cylinder assembly plant number match before the strand is inserted in the assembly. The sample number and its assembly number are then noted and the duck tape from both ends of the strand is removed. The strand is then inserted into the can. Check to make sure that the bottom end of the strand touches the wooden form. Steel aligners also called “tops” are then placed on the specimen to avoid any possible movement of strand during casting and to ensure uniform filling of mortar. They also ensure that the strand is at the centre of the cylinder during casting.

Batching:

The bond strength of strand wire is tested in cement mortar. In order to achieve this compressive strength trial batching is carried out. Trial batching is carried out every time a new batch of cement or sand arrives. This is because every shipment of sand and/or cement has different properties. The proportions of cement, sand and water need to be adjusted based on these properties, to achieve the specified compressive strength. The density, specific gravity and absorption of sand are tested in accordance with ASTM C128-01. The fineness of sand is tested in accordance with ASTM C 136-01.

For the purpose of trial batching, four water cement (w/c) ratios (.45, .48, .51 and .54) were chosen and nine 2in mortar cubes are cast per batch. The mean of the cube strengths per batch are then plotted against the w/c ratios. From this plot, a w/c ratio at which the compressive strength falls within the permissible range is obtained.

With the w/c ratio determined, production batching can be carried out. However, by the passage of time the sand in storage looses moisture and thus the moisture content of sand should be tested in accordance with ASTM C566-97 before every batch. Proportions of water and sand in the mix should be adjusted based on the moisture content of sand. Once the proportions of ingredients are calculated they are weighed and kept in readiness.

Mixing Process:

The concrete mixer is started and the ingredients are added in the following sequence;

Firstly, half the sand was put in the mixer pan, then half the water. Secondly, all the cement and the remaining water. Lastly, the remaining sand in the bucket and covered the mixer. The time for the process was noted as soon as the cement hit the water.

The mixer is first run for 3 minutes and stopped. Paddles should then be raised and cleaned using a trowel. The paddles should then be lowered and kept resting for 3 min. The 3 min is counted from the time the mixer is stopped. After a three minute resting period the mixer should then be run for 2 minutes and stopped. All the mixing was done as per ASTM C 192.

The flow of the mix should then be measured in accordance with ASTM C 1437-01. The flow of mortar should fall in the range 100-125 as .Specimens should be cast only if the flow falls in this range. If the mortar is less than 100, then the mixing time is increased to one more minute until the required flow is achieved. If the required is not achieved then the batch mix is discarded. The Unit weight was also measured. If the unit weight of the fresh mortar is not close to the unit weight measured routinely during trial batching or previous mixtures, the fresh mortar is generally discarded.

Figure 4.22: NASP Specimens ready to be batched.

The test specimens should be filled with mortar in two lifts; filled halfway, then vibrated, filled to the top and then vibrated. While vibrating in the first lift, the vibrator should not touch the plate or the sides of the cylinder assembly. While vibrating in the second lift, the tip of the vibrator should not go below the upper half of the assembly. After vibration in the second lift, the cylinder assemblies need to be tamped on four sides using a rubber hammer to expel air voids and close the gap created by the vibrator. Due to vibration, the mortar in the cylinder assembly settles. The remainder of the specimen, after vibration, should be filled with mortar up to the top and finished. Mortar when fresh is easier to handle than after it is set.

Thus to reduce the time of specimen preparation before testing, sides of the specimens and tops should be cleaned to remove excess mortar when fresh.

While specimens are being cast, the mortar cubes should be cast simultaneously based on ASTM C 109/C 109M-02. After the batching process is complete the specimens and the cube moulds should be transported to the curing room and kept there for 22hrs. The curing room temperatures follow ASTM C 511 – 98.

Testing:

Testing is carried out in the four hour window between 22-26 hours measured from the time the cement hits the water. Before the specimens are to be tested they need to be prepared for testing. This preparation takes any where between 45mins to an hour. Thus we need to start preparing the specimens at approximately 21hrs. To ensure that the specimens are cured for at least 22hrs, most of the specimen preparation should be carried out in the curing room.

4.3.10 PREPARING SPECIMENS FOR TESTING

This process should start at 21hrs. The tops of the specimens should be removed and the exposed strand and sides of the cylinder assembly are to be cleaned with a wire brush**.** This is done to ensure that;

1. The magnetic bases of the LVDT and it's support assembly fit firmly onto the face of the steel cylinder

2. The aluminum plate from which slip is measured, sits on the king wire. Thus the specimens are ready for testing.

Preparing the cubes for testing.

The process of preparing cubes should start after the specimens are prepared for testing. This will give the cubes more time in the curing room. Every mould contains 3 cubes, which are tested for compression. The exposed surface of the cubes should be sanded and brought in level with the mold. Then the top and bottom surface areas that will be subject to compression should be measured using vernier calipers. The area under compression should then be calculated. Thus the cubes are ready for testing.

Testing Procedure:

Following is a stepwise description of the testing procedure

The hydraulic actuator was warmed up for 10 to 15 minute.

The following were checked in the controller settings. The controller used was the Test Star s on an MTS Testing Set –Up.

The results of the test are valid only if the mortar strength is in the range of 4500-5000psi. Thus at 22hrs the first cube is tested. If the mortar has attained the required strength, the next two cubes are also tested. If the compressive strengths fall within the required range, testing of the specimens was started. If the mortar had not achieved the desired strength in 22 hrs, a break of half an hour was taken and one cube was tested again to measure the strength. If we are testing 12 specimens and the mortar had not achieved the required strength at 24hrs, the test should be repeated. This is due to the 22-26 hrs time constraint.

Bring the internal LVDT in the actuator to a level at which we can fit the specimen in the frame and fit the pre-stressing chuck on it. This position will be the zero point for testing.

The specimen should then be mounted on the testing frame with the neoprene pad and base plate as shown in the Figure below:

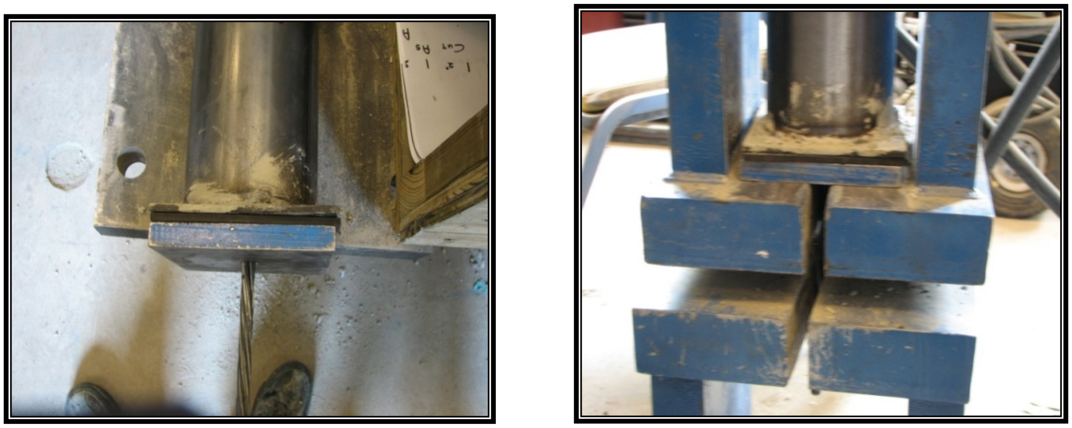


Figure 4.23 : Position of Neoprene Pad, Base Plate Specimen is mounted on the Test Frame

After the specimen is mounted on the testing frame, the base plate for the pre-stressing chuck and the pre-stressing chuck assembly should be attached to it. The chuck should be paced such that the three jaws or the chuck are in the same plane The external LVDT support system

Figure 4.24: Levelling the External LVDT

should then be attached and leveled as shown.

After the support system is leveled, the external LVDT should be attached and leveled as shown in Figure .

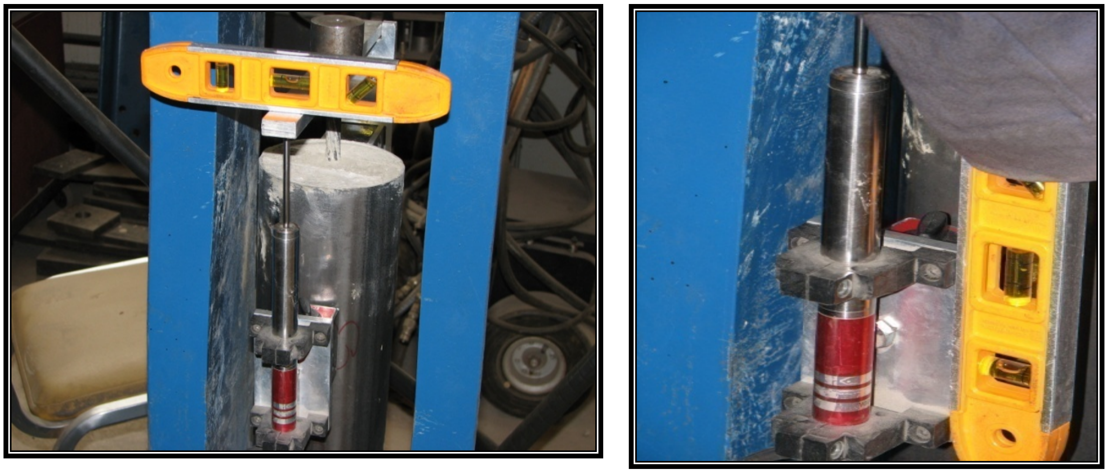


Figure 4.25 : Leveling of External LVDT.

Run the test and the load for .01 in. axial slip and .1in axial slip” were noted.

Figure 4.26: NASP Test Set Up for Bond Test

4.4 Test Procedures for NASP Test Protocols:

4.4.1 Mortar Strength:

The variations in the mortar strength were brought about by varying the w/c. Different mortar strengths were tested were as with w/c 0.40,0.45 and 0.50 . According to Kiran (2006), two different types of strands were selected initially to study the effects mortar strengths c on the NASP pull out values. The 2001 NASP protocols attached in Appendix had a range of mortar strengths from 3500 to 5000 psi. The wide spread in the mortar range posed a larger spread in the NASP values. It was seen that the mortar strength had significant effects towards the NASP pull out values. Studies conducted in this research helped in identifying a closer range of mortar strengths which in turn resulted in more consistent NASP values. However, a closer work on the mortar strength showed that the mortar strength in the range of 4750 – 5000 psi , resulted in more consistent NASP values.

4.2 Load Control V/S Displacement Control.

The 2001 NASP test protocols are attached in the Appendix with the specification. According to Kiran (2004), a displacement control of 0.1in/min was suggested as per the refinement to the test methodology. All the studies were conducted to understand the effects of controlling the NASP test in load control and displacement control. The load control of 5000 lb/min and a displacement control of 0.1in/min were performed on the NASP specimens.

He found that the displacement controlled loading rate and the load controlled loading rate showed significant differences in the NASP test. The strands when tested in the Displacement control had showed a “softening nature”. The NASP pull out value seen for load controlled testing was higher than the same tests conducted with the displacement control (Kiran, 2004).

Importance of loading rate:

Studies compiled in earlier research work (Grieve 2004) reported significance of loading rate on the NASP bond test. The studies were not performed using the present NASP protocols. The Strands used in the study were the NASP strand “C” and “G”. also called as “FF” and “AA” respectively as per NASP Round III. The Fig showed the variation of the loading rate on the NASP values.

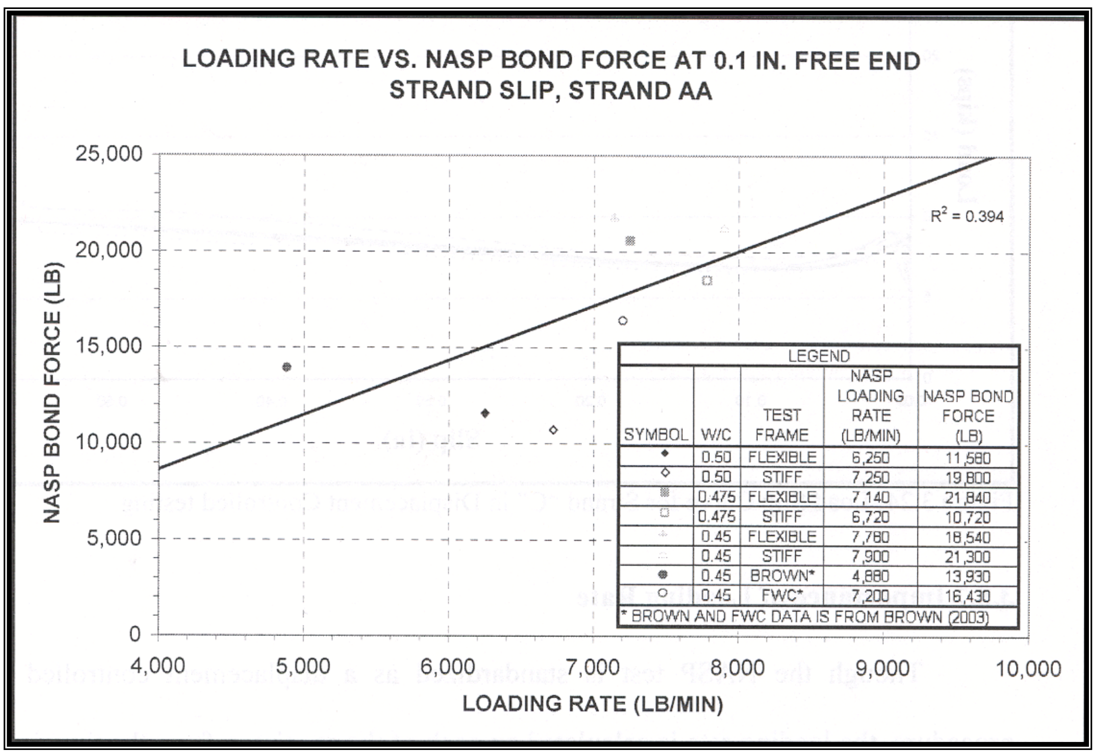


Figure 4.27 : Loading Rate and NASP Bond Force for Strand AA (Grieve 2004)

Grieve (2004) reported that, “Based on the regression analysis, the correlation between NASP bond forces and loading rate is relatively small when all the data is considered.

Although the correlation does increase when looking at the OSU data only, the data does not support a strong relationship between the loading rate and NASP bond force."

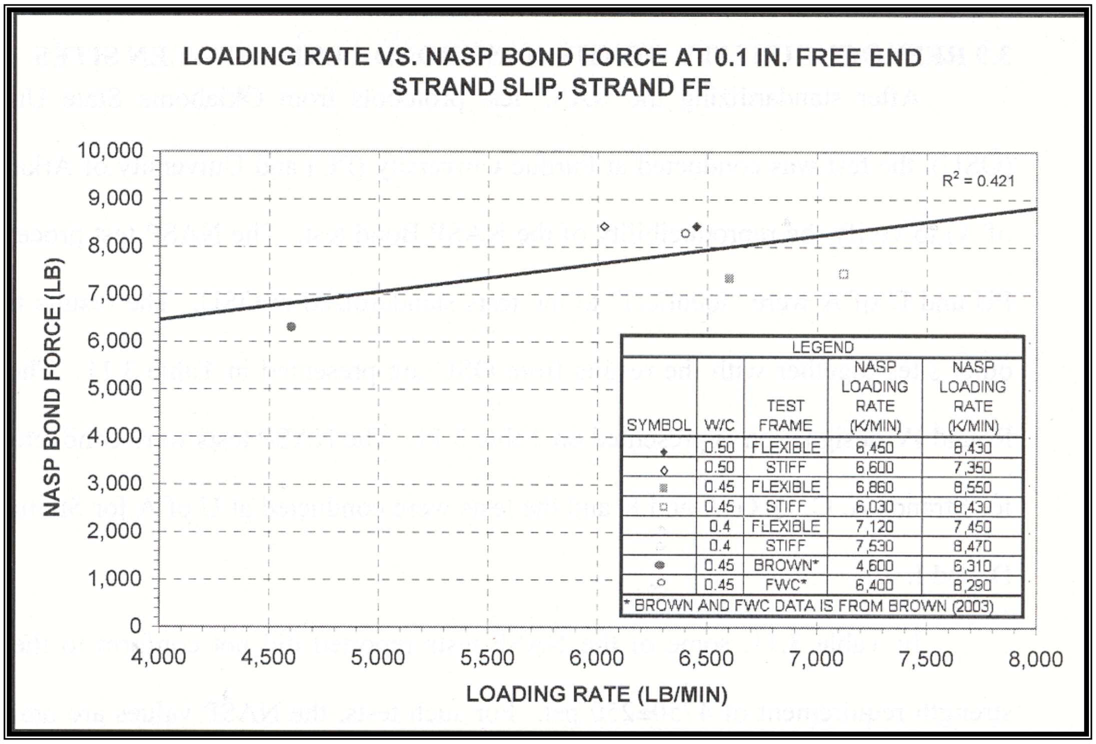


Figure 4.28: Loading Rate and NASP Bond Force for Strand FF (Grieve, 2004)

In the current NASP test protocol, the maximum loading rate for a specimen during the NASP test was limited to 8000 lb/min. While conducting the test at OSU using the NASP protocols mentioned in the earlier section, all the loading rates were within 8000 lb/min when computed from time increment between 4000 and 6000 lbs.

4.4.3 Mortar Flow:

Earlier the mortar flow was not specified in the NASP 2001 protocols, however this was a very important factor to understand the consistency of the mix. With the increase in the amount of water in the mix the flow tends to be higher. The flow increased with the w/c. The measured flow decreased significantly over time in the fresh state from one trail to another (Kiran, 2004). The flow measurements were made in conformance with ASTM C 1437. In order to have a consistent mortar mix, it is necessary to have a flow range. By various experimentations and trial batches suggested by Kiran (2004), a flow range of 110 to 125 was specified to NASP test protocols to achieve a consistent mix proportions in the NASP specimens. A flow which was out of range from the mix proportions indicated a problem with the mixture constituents.

4.4.4 Curing Temperature:

As the 24 hour compressive strengths had considerable effects with the curing temperature, trial batches were made to study the optimal curing temperature range for NASP specimens (Kiran, 2004). The compressive strengths (in psi) of the individual mortar cubes with varying temperature were tested. The ASTM curing conditions were maintained in the laboratory curing room with 70 to 73 F curing temperatures. It was found that the compressive strength increase with the increase in curing temperatures. Thus , it was concluded that to achieve the compressive strength between 4750+/-250 psi, the curing temperature should be maintained at 70+/-3F.

4.4.5 Loading Rate:

According to Chandran (2004), though the applied loading was displacement control, the loading rate of the test was calculated. The loading rate was reported for all the NASP tests as the rate of lbs. per minute when the axial load on the specimen is between 4000 to 6000 lbs.

Loading Rate, lb/min = (P@4000 – P@6000)/(t@4000 – t@6000)*60

Where,

P@4000 and P@6000 are the exact loads at 4000 and 6000 lbs and

t@4000 and t@ 6000 are the respective time in seconds.

4.5 RESULTS OF NASP BOND TEST:

The tables below shows the NASP pull out values for the Strand "C" when it was tested in the mortar grout. The range of the mortar strength reported for NASP was from 4500 to 4750 psi. There were 45 individual data points that were tested both against the strand "C" and strand "A". The various parameters like Fineness, Fine to coarse ratio, Flow, Mortar strength and Fresh Unit wt were analyzed by regression analysis.

Strand "C"

Date	Cylinder #	Axial Pullout Strength (lbs) @ 0.1" Slip	Fineness Modulus of F.A.	F.A. to Cement Ratio	Flow	Mortar Strength (psi)	Fresh Unit Weight (pcf)
9/27/2006	C-1	5600	2.56	2.05	127	4709	134.77
(Q3 2006)	C-2	6020	2.56	2.05	127	4709	134.77
10/17/2006	C-1	6230	2.56	2.05	121	4673	134.77
(Q3 2006)	C-2	6650	2.56	2.05	121	4673	134.77
	C-3	7090	2.56	2.05	121	4673	134.77
11/3/2006	C-1	8310	2.56	2.05	124	4877	134.66
(Q4 2006)	C-2	7310	2.56	2.05	124	4877	134.66
	C-3	7530	2.56	2.05	124	4877	134.66
12/8/2006	C-1	7590	2.56	2.15	130	4838	135.88
(Q4 2006)	C-2	7770	2.56	2.15	130	4838	135.88
	C-3	7550	2.56	2.15	130	4838	135.88
12/19/2006	C-1	7970	2.55	2.50	129	4790	136.76
(Q4 2006)	C-2	7450	2.55	2.50	129	4796	136.76
	C-3	8320	2.55	2.50	129	4796	136.76
12/20/2006	C-1	8460	2.55	2.50	110	4616	133.23
(Q4 2006)	C-2	8190	2.55	2.50	110	4616	133.23
	C-3	7680	2.55	2.50	110	4616	133.23
12/21/2006	C-1	8050	2.55	3.50	84	4559	134.55
(Q4 2006)	C-2	8320	2.55	3.50	84	4559	134.55
	C-3	9430	2.55	3.50	84	4559	134.55
2/13/2007	C-1	7960	2.55	3.00	119	4668	134.00
(Q1 2007)	C-2	7370	2.55	3.00	119	4668	134.00
	C-3	8220	2.55	3.00	119	4668	134.00
	C-4	8550	2.55	3.00	119	4668	134.00
5/1/2007	C-1	9850	2.81	3.00	125	4899	137.42
(Q1 2007)	C-2	9530	2.81	3.00	125	4899	137.42
	C-3	9270	2.81	3.00	125	4899	137.42
	C-4	9190	2.81	3.00	125	4899	137.42
	C-5	9500	2.81	3.00	125	4899	137.42

	C-6	10360	2.81	3.00	125	4899	137.42
6/8/2007	C-1	8940	2.81	3.02	119	4870	137.75
(Q2 2007)	C-2	9260	2.81	3.02	119	4870	137.75
	C-3	8460	2.81	3.02	119	4870	137.75
	C-4	10190	2.81	3.02	119	4870	137.75
	C-5	10030	2.81	3.02	119	4870	137.75
	C-6	10080	2.81	3.02	119	4870	137.75
6/20/2007	C-1	7910	2.81	3.02	119	4624	137.75
(Q2 2007)	C-2	9090	2.81	3.02	119	4624	137.75
	C-3	8760	2.81	3.02	119	4624	137.75
	C-4	10450	2.81	3.02	119	4624	137.75
	C-5	8500	2.81	3.02	119	4624	137.75
	C-6	9670	2.81	3.02	119	4624	137.75
8/17/2007	C-1	9590	2.81	3.02	118	4773	137.42
(Q3 2007)	C-2	10370	2.81	3.02	118	4773	137.42
	C-3	10460	2.81	3.02	118	4773	137.42
	C-5	11040	2.81	3.02	118	4773	137.42

4.6 Regression Analysis

In order to quantify the various parameters that affect the bond strength of the

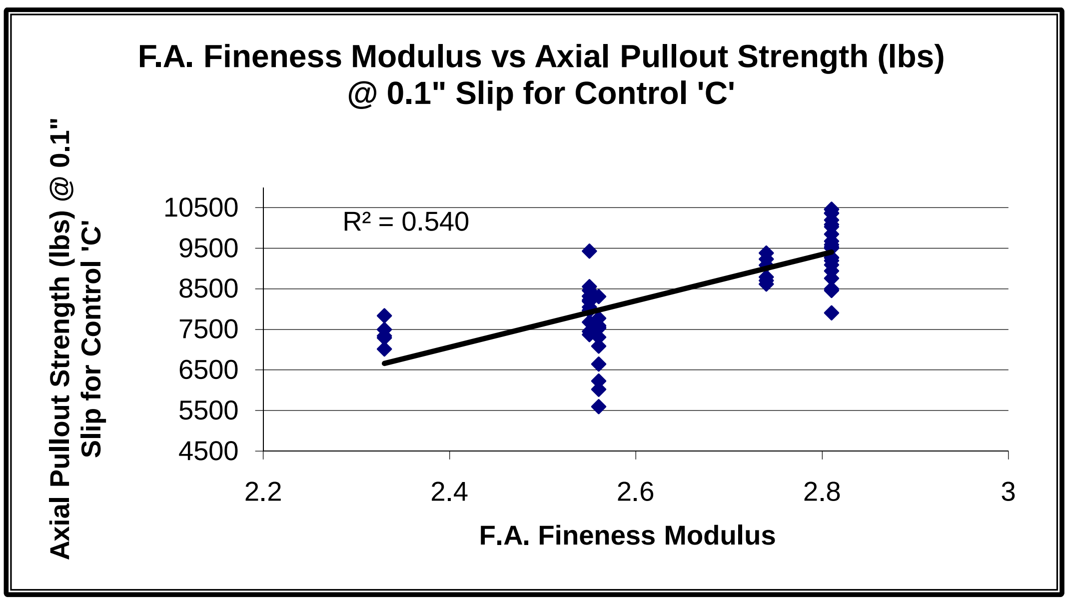


Figure 4.29: Linear Regression of F.A, fineness modulus v/s bond strength of Strand "C".

Pre-stressing strands linear regression plots were plotted against the test variables. The fineness modulus of sand has a higher correlation with the bond strength for strand "C" than other parameters. Thus the mortar strength, mortar flow and unit weight have hardly any effect on the bond. To conclude if the fineness of sand is kept in a fixed range, then the test can is very much valid to be as a Standard Test for Strand Bond.

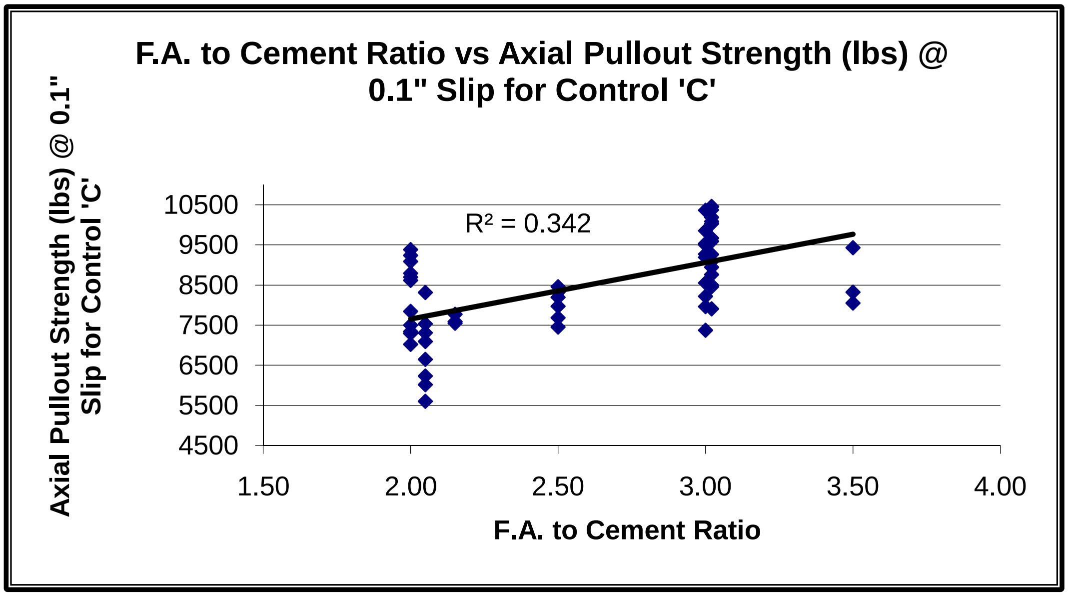


Figure 4.30 : Linear Regression of F.A/Cement v/s Bond strength of Strand "C".

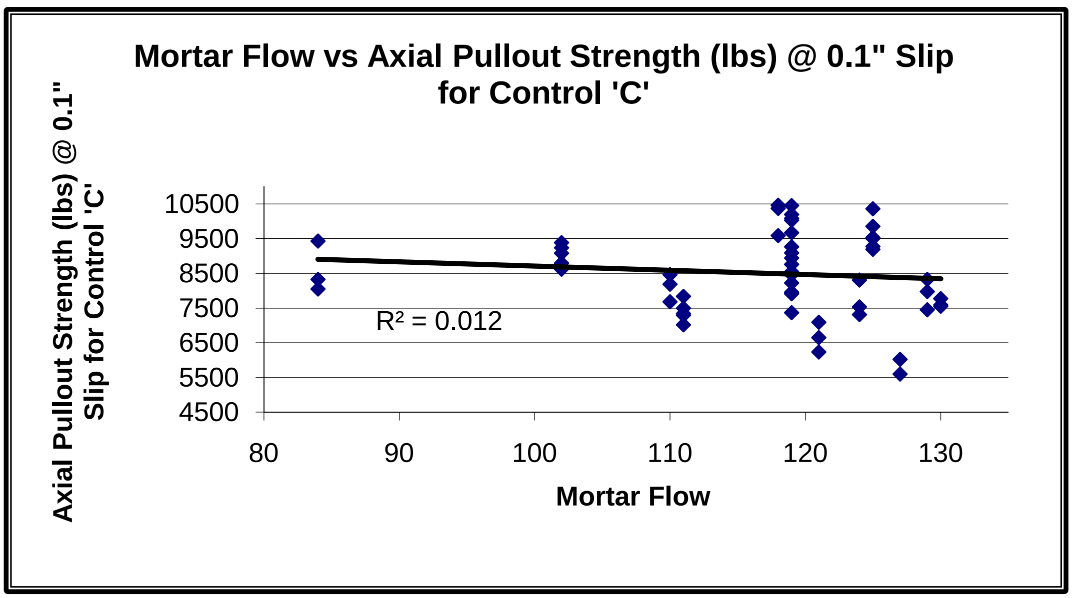


Figure 4.31: Linear Regression of Mortar Flow v/s Bond Strength of Strand "C".

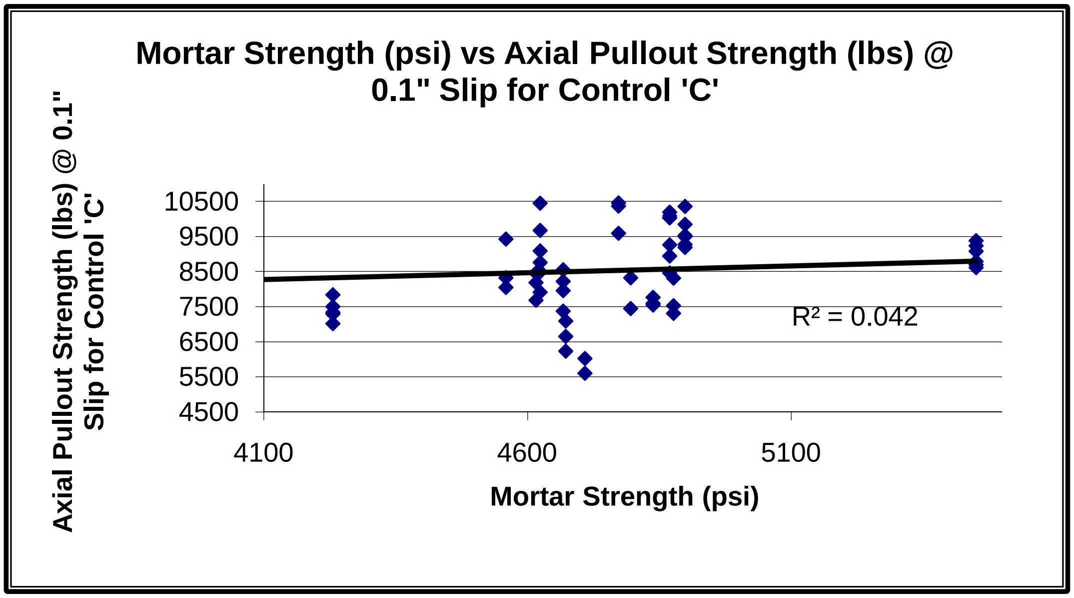


Figure 4.32: Linear Regression of Mortar strength v/s Bond Strength of Strand “C”.

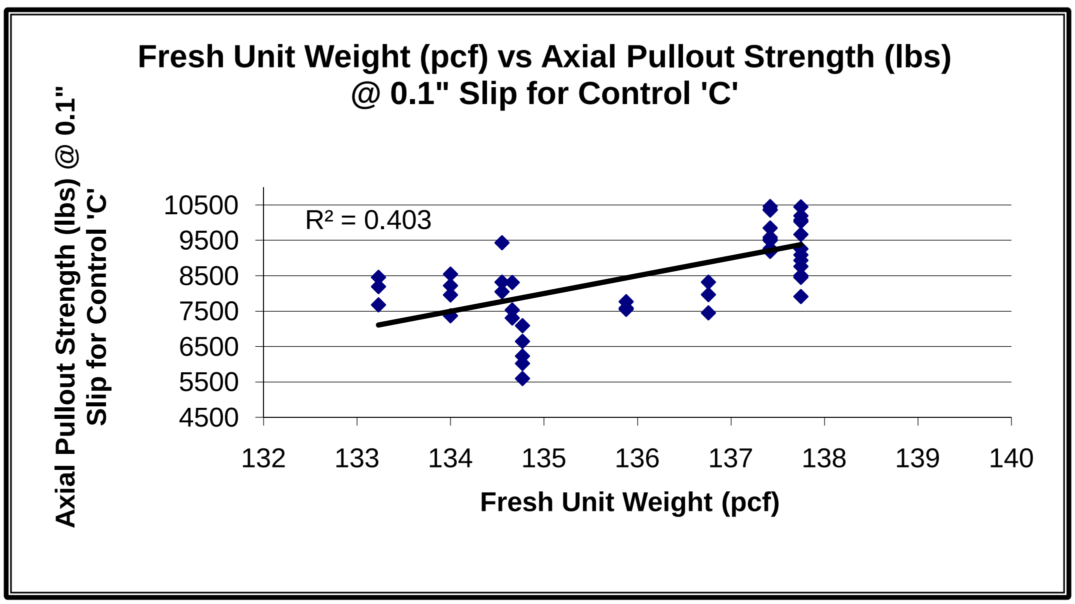


Figure 4.33 : Linear Regression of Fresh Unit wt v/s Bond Strength of Strand “C”.

Strand "A":

Similarly, the Strand "A" variation was plotted against various variables, like the F.A of sand, F.A to cement ratio, Flow, Mortar strength and Fresh Unit wt. The table below summarizes all the variables as listed.

Date	Cylinder #	Axial Pullout Strength (lbs) @ 0.1" Slip	Fineness Modulus of F.A.	F.A. to Cement Ratio	Flow	Mortar Strength (psi)	Fresh Unit Weight (pcf)
10/17/2006	A-1	13850	2.56	2.05	121	4673	134.77
(Q3 2006)	A-2	15740	2.56	2.05	121	4673	134.77
	A-3	15870	2.56	2.05	121	4673	134.77
11/3/2006	A-1	15700	2.56	2.05	124	4877	134.66
(Q4 2006)	A-2	14150	2.56	2.05	124	4877	134.66
	A-3	15710	2.56	2.05	124	4877	134.66
12/8/2006	A-1	20920	2.56	2.15	130	4838	135.88
(Q4 2006)	A-2	19900	2.56	2.15	130	4838	135.88
	A-3	19120	2.56	2.15	130	4838	135.88
12/19/2006	A-1	17080	2.55	2.50	129	4796	136.76
(Q4 2006)	A-2	16520	2.55	2.50	129	4796	136.76
	A-3	15950	2.55	2.50	129	4796	136.76
12/20/2006	A-1	20680	2.55	2.50	110	4616	133.23
(Q4 2006)	A-2	18760	2.55	2.50	110	4616	133.23
	A-3	16000	2.55	2.50	110	4616	133.23
12/21/2006	A-1	18700	2.55	3.50	84	4559	134.55
1/20/2007	A-1	18610	2.55	3.00	116	4598	135.88
(Q4 2006)	A-2	15750	2.55	3.00	116	4598	135.88
	A-3	18850	2.55	3.00	116	4598	135.88
2/13/2007	A-1	18480	2.55	3.00	119	4668	134.00
(Q1 2007)	A-2	18580	2.55	3.00	119	4668	134.00
	A-3	17710	2.55	3.00	119	4668	134.00
	A-4	18210	2.55	3.00	119	4668	134.00
4/27/2007	A-1	15020	2.81	3.00	123	4894	137.31
(Q1 2007)	A-2	19630	2.81	3.00	123	4894	137.31
	A-3	18500	2.81	3.00	123	4894	137.31
	A-4	18560	2.81	3.00	123	4894	137.31
	A-5	16460	2.81	3.00	123	4894	137.31
	A-6	16540	2.81	3.00	123	4894	137.31
6/7/2007	A-1	16410	2.81	3.02	115	4768	138.52
(Q2 2007)	A-2	15360	2.81	3.02	115	4768	138.52
	A-3	16950	2.81	3.02	115	4768	138.52

	A-4	16760	2.81	3.02	115	4768	138.52
	A-5	16670	2.81	3.02	115	4768	138.52
	A-6	17080	2.81	3.02	115	4768	13852
6/21/2007	A-1	17350	2.81	3.02	121	4683	137.42
(Q2 2007)	A-2	19100	2.81	3.02	121	4683	137.42
	A-3	19970	2.81	3.02	121	4683	137.42
	A-4	16200	2.81	3.02	121	4683	137.42
	A-5	19200	2.81	3.02	121	4683	137.42
	A-6	17890	2.81	3.02	121	4683	137.42
8/16/2007	A-1	15630	2.81	3.02	118	4735	137.42
(Q3 2007)	A-2	19370	2.81	3.02	118	4735	137.42
	A-3	19020	2.81	3.02	118	4735	137.42
	A-4	14750	2.81	3.02	118	4735	137.42
	A-5	16800	2.81	3.02	118	4735	137.42
	A-6	17230	2.81	3.02	118	4735	137.42

Similarly, the correlation between the various factors like fineness of F.A, Fine to cement ratio, mortar strength and unit weight are plotted to understand the bond strength on Strand “A”.

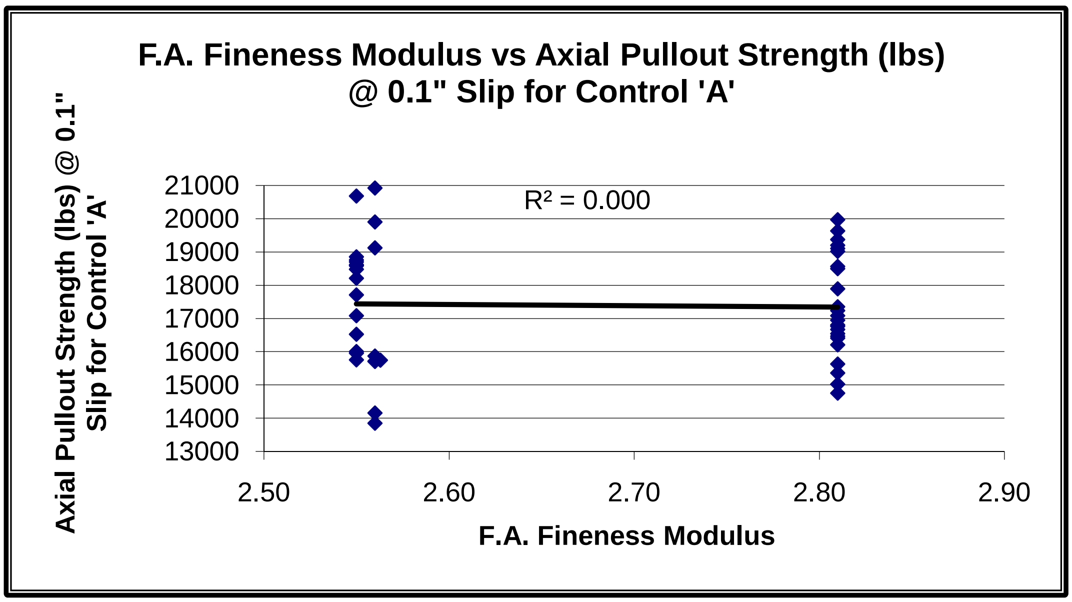


Figure 4.34: Linear Regression of F.M of F.A on Bond Strength of Strand “A”.

In contrast for higher performing strands like Strand “A”, there are hardly any effect of fineness modulus, fine to cement ratio, mortar flow, mortar strength and fresh unit weight on the bond strength. The regression analysis also proved that these factors have no correlation on strand bond, when the strength of the mortar cubes are in the range of 4400 to 5000 psi as per the test specifications. Thus this test can be stated as the Standard test for Strand Bond.

The figure below is a plot of linear regression against the axial pull values (0.1" slip) and the fine to cement ratio.

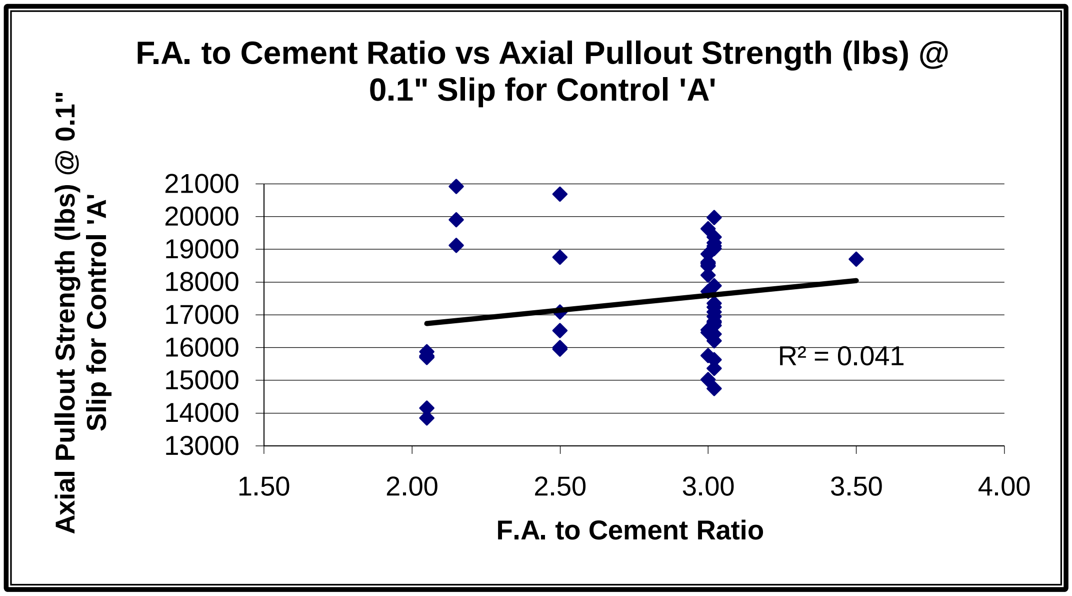


Figure 4.3 : Linear Regression Analysis of F.A/Cement to Bond Strength strand "A"

The regression plot showed there was no regression between the pull out values and the fine aggregate to cement ratio.

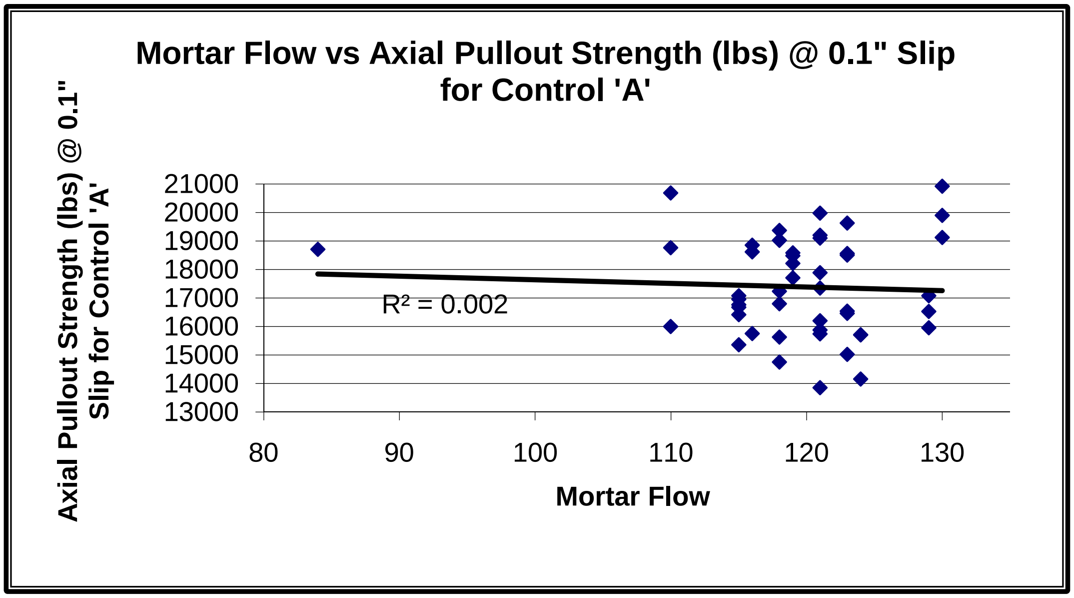


Figure 4.36: Linear Regression Analysis of Mortar Flow v/s Bond Strength of Strand “A”

The plot above

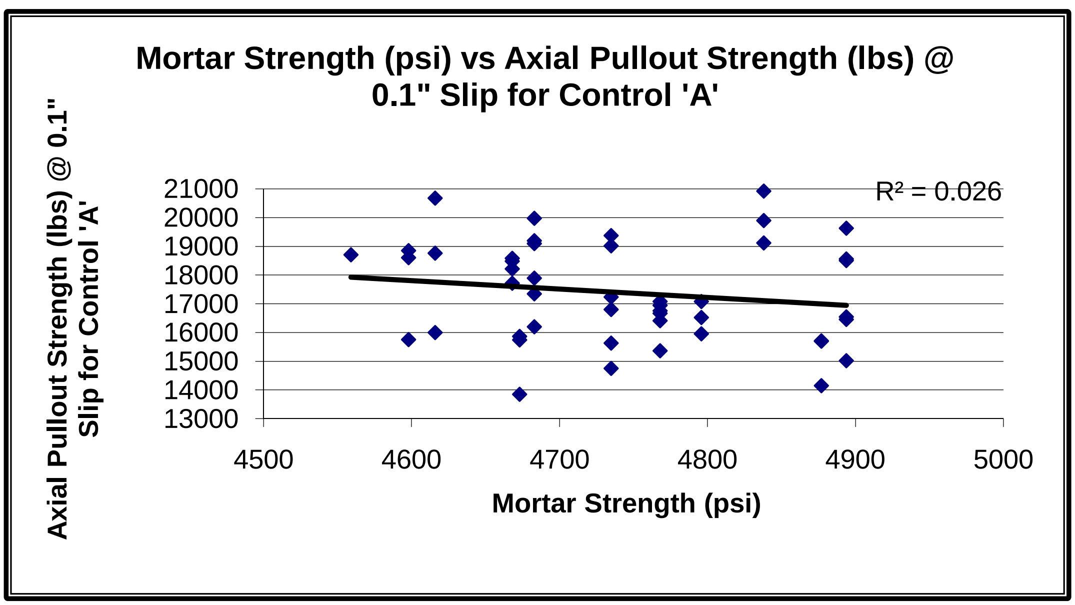


Fig.4.37 Linear Regression Analysis of Mortar Strength v/s Bond Strength for Strand “A”

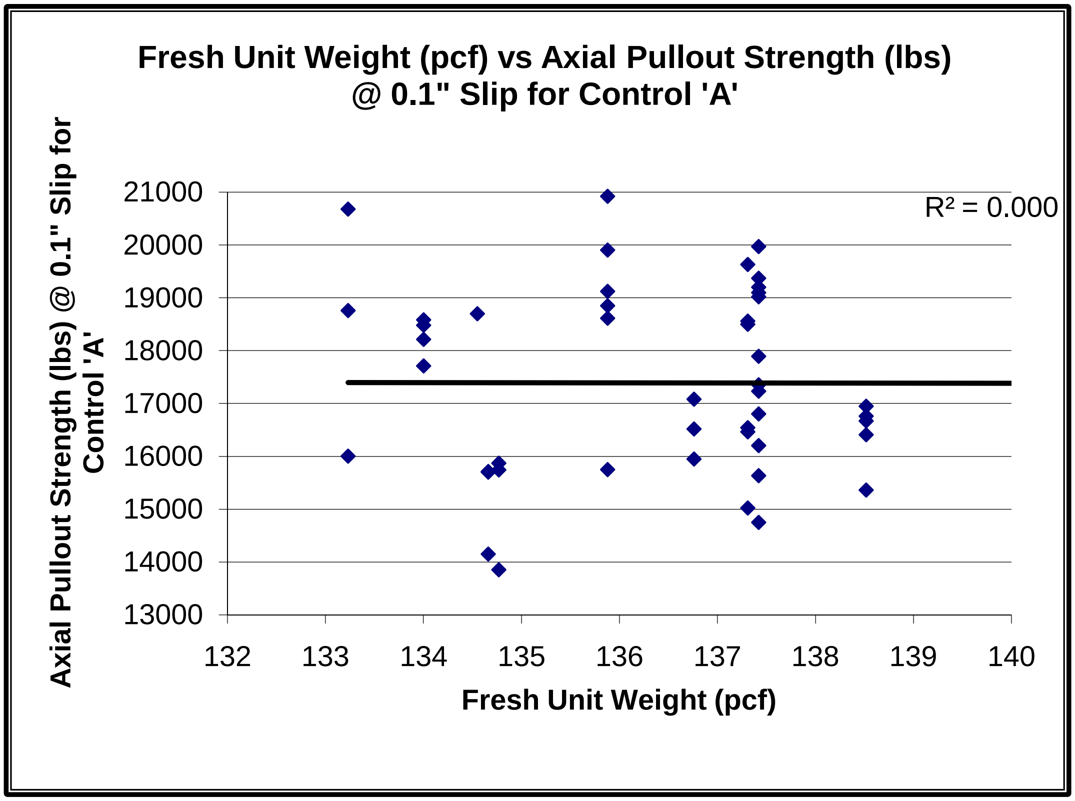


Fig 4.38: Linear Regression Analysis of Fresh Unit Weight v/s Bond Strength of Strand "A".

4.7 Summary of Test Results:

Control Strand	Number of Data Points	Strand Slip (in)	Mean (lbs)	STDV (lbs)	95% Confidence Interval (+/- lbs)	Coef. of Variation (%)
C	45	0.01"	8687	806	240	9.27
	45	0.1"	8568	1268	378	14.79
A	46	0.01"	11556	1504	444	13.02
	46	0.1"	17389	1726	509	9.93

4.8 Conclusion of Test Results

The table above summarizes the test results of the bond test on strands with the mortar grout. The numbers of observations were 45 for both the groups. The standard deviation of the group for Strand "C" for 0.1 in slip was 1268 lbs, mean was 8568 lbs and the coefficient of variation was only 14.79%. Since the coefficient of variation was low, it can be said that the test was very uniform. Similarly, for strand "A" the standard deviation was 1726 lbs, mean 17389 lbs and coefficient of variation of 9.93%. Thus, it can be very well accepted as a "Standard Test for Strand Bond".

CHAPTER V

NASP TEST ON SELF-CONSOLIDATING CONCRETE

The NASP Bond test protocol was modified to test the strand in self-consolidating concrete in place of mortar. This chapter will discuss and report the results of the NASP test conducted in SCC. The modified NASP test in SCC was performed on two 0.5 in diameter (12.7mm). The major research variables are the coarse aggregate content and size in the three different mixes. The w/c and the other variables like admixture dosage rates are kept constant. The SCC mixes SCC1, SCC 2, SCC 3 and C-N are tested for NASP bond test. All the test samples are kept in the specified compressive strength as designated by the NASP protocols in the previous chapter.

5.1 Scope of Research:

The modified NASP test was conducted in SCC in order to understand the effects Of varying the coarse aggregate content with respect to the fines on the bond of pre-stressing strands. The test procedure was identical to the NASP Test protocols as discussed in Chapter 3, except SCC was used instead of the cement mortar. The NASP tests in concrete were conducted on two different strand sources of 0.5 in dia. The target concrete strengths were 4750 to 5000 psi.

5.2 Research Variables:

The standard NASP bond test procedure was performed on cement mortar as explained in Chapter 3. However, the test protocols do not depend on the tests conducted in SCC. This test was conducted in SCC to study the effect it has on varying mixture proportions like reducing coarse aggregate sizes. The dosage of the admixtures were not the variable used here as all the HRWRA , VMA and Retarders are kept constant in all the SCC mixes. The w/c cement ratios are kept same in all the SCC mixes so that the effect of pre-stress bond can be evaluated with respect to the different content of coarse aggregate and with the normal conventional concrete. The concrete strengths used for the modified NASP test was kept in the range of 4750 to 5000 psi.

Material properties:

Pre-stressing strands:

All the pre-stressing strands used for this research program were seven wire Grade 270 ksi low relaxation strands from manufacturers in North America. The strands conformed to ASTM A 416 attached in the Appendix J. The pre-stressing strands had a nominal diameter of 0.5 in and area of 0.153 sq in. The modulus of elasticity of the pre-stressing strands was estimated as 28,500 ksi (196.3 Gpa) for 0.5 in diameter strands.

Self-Consolidating Concrete Mixtures:

The concrete mixtures used for making the NASP specimens in the concrete included the Type III cement from Buzzi Unicem USA as per the ASTM C-150 Specifications for Portland Cement, coarse and fine aggregate from Dolese Bros. Co and admixtures from Grace chemicals. Admixtures included High Range water reducing admixtures, viscosity modifying admixtures and retarders. The table below summarizes the mixture proportions and the target fresh and hardened properties for the SCC cast in the modified NASP specimens. The Normal Concrete C-N was made the basis of the test from which all the other SCC mixtures were modified depending on the research specifications. The mixture details were tabulated as follows:

BOND TEST ON SCC:				
COMPONENT	**NC**	**SCC -1**	**SCC - 2**	**SCC - 3**
		SSD WTS (lb/cyd)		
CEMENT (TYPE -3)	648	745	745	778
FINE AGGREGATES	1146	1128	1549	1393
COARSE AGGREGATES	1722	1581	1264	1257
WATER	298	350	350	350
W/C RATIO	0.46	0.47	0.47	0.45
HRWRA - ADVA CAST – 575 (fl oz)	0	10	10	10
VMA - VMAR 3 (fl oz)	0	30	30	30
RECOVER (fl oz)	0	1	1	1
MIXTURE DESIGN PROPERTIES:	**C-N**	**SCC-1**	**SCC - 2**	**SCC -3**
Fresh Concrete Properties				
Slump flow spread (in)	8	25.15	27	26.15
Inverted Slump Flow (in)	-	23.7	24.6	23.9
T 20 seconds	-	3.3	2	2
J- ring (in)	-	0.43	0.13	0.62
L- Box Filling Head Drop (H1/H2) in		1.2	1.1	1.0
Air Content (% by vol)	1.5	0.5	0.2	0.8
Unit weight (pcy)	149	148	146	147
Temperature ©	24	30	25	25
Water/Cement Ratios (w/c)	0.46	0.47	0.47	0.45
Hardened Concrete Properties				
Compressive Strength (Type 3)				
1 day (psi)	4994	4615	4443	4866
7 day (psi)	6844	6140	6048	6697
28 day (psi)	7792	7083	6685	8256

Table 5.1: Summary of Parameters on Bond Test on SCC

5.3Procedure for NASP Bond Test:

The NASP Bond Test tests the bond quality of the 0.5 in pre-stressing

Strands that conform to the ASTM A 416. The test was carried out by casting the pre-stressing strands in SCC enclosed in a cylindrical steel form with a base plate. The mix proportions used for conducting the NASP in SCC were explained in detail in the previous table. The strands were pulled out from the concrete at a loading rate of 0.1in/min using a hydraulic system after curing for 24+/-2 hrs. The pull out force was measured with respect to the free end movement of the strand to the specimen. The NASP bond test recorded the pull out force that corresponded to the 0.10 in of free end slip. As explained in the Chapter 3, each single NASP bond test consisted of six individual test specimens. The NASP bond value was the average of the of the six bond slip values.

5.4.1Test Methodology and Sample Preparation:

Sample Preparation:

The NASP Bond Tests conducted in SCC were used as the same procedure as explained in the previous chapter. The preparation of the strands , steel casing and the bond breakers were identical to the NASP tests conducted in sand cement mortar. The diagram below shows the sample before it is ready to be batched.

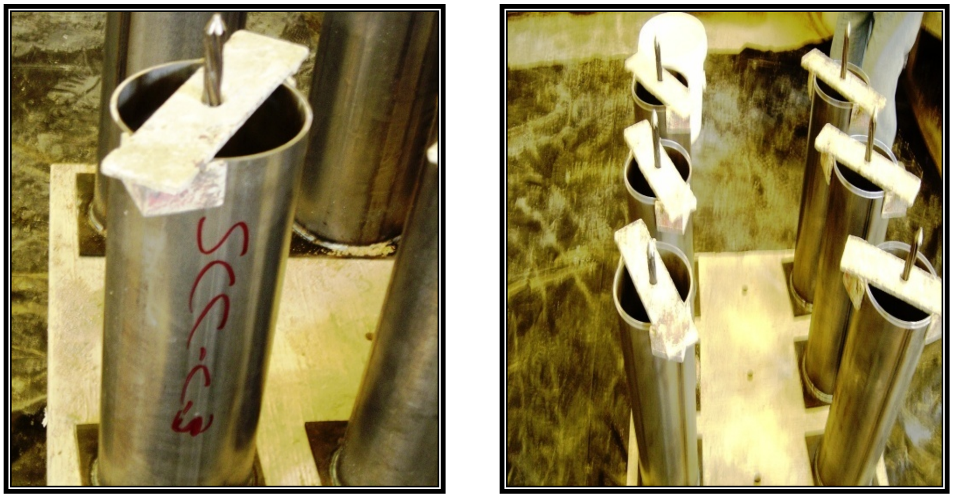


Figure 5.1 : SCC-3 ready to be Bond Tested.

Concrete Batching :

The concrete batching was conducted in a pan mixer. The mixing procedures used NASP Bond test conformed to ASTM C 192. The fresh SCC was filled in two major lifts. After filling half the layer, the cylinders were tamped at its side to free of the entrapped air. No mechanical vibrator was used as the concrete used to consolidate of its own weight. Then rest half of the cylinders was filled and tamped to remove the entrapped air. The slump, unit weight and air content are measured as per ASTM C 143, ASTM C 138 and ASTM C 231 respectively. The figure shows the filling of SCC mixes in the cylinders.

Figure 5.2: SCC ready to be filled in the NASP specimens.

Figure 5.3: SCC scooped and filled in the NASP specimens.

The NASP specimens and the test cylinders were cured in conformance with ASTM C 192. The compressive strength testing was conducted during the time of the NASP test, in conformance with ASTM C 39. The NASP specimens are then kept in the curing room for 22 to 24hrs where the temperature was maintained at 74 F and relative humidity 100%.

Figure 5.4: After curing SCC-2 ready to be tested for NASP bond Test.

5.5 NASP Test in SCC:

The modified NASP bond test in SCC was performed at 22 to 24hrs after the hydration of the cement. The NASP specimen in concrete was mounted on a rigid steel frame in the same manner described for the NASP Bond Test in Mortar as in the previous chapter.

Figure 5.5: NASP Bond Test in SCC -2 as per the new Test Set-Up.

5.6 NASP Results in Self-Consolidating Concrete:

The results from this experimental testing are summarized along with the NASP value of the strand in the standardized NASP test. The concrete strengths reported are the average strength of three or more concrete specimens tested during the NASP test. The number of specimens tested (N) and the standard deviation (S) are reported for the modified NASP test in SCC. The sample size was small in this testing case.

Concrete Type	Strand Type	Testing Date	STSB Tests Mean Concrete Strength (psi)	Pull Out Force at 0.1" Slip	STD Dev	COV (%)	w/c
C-N	C(0.5)	24-Jun-08	4259	8850	774	8.74	0.46
SCC -1	C(0.5)	28-Jun-08	4368	7625	1197	15.70	0.47
SCC -2	C(0.5)	4-Jul-08	4403	9633	1293	13.42	0.47
SCC -3	C(0.5)	30-Jun-08	4192	9725	1093	11.24	0.45

Table 5.2: Summary of NASP Test Results in Strand “C” (0.5)

The standard deviations for SCC- 3 and C-N are less than other mixes. This means that with less rock and more fines, the pull out force were more consistent than other mixes, though the sample size was small.This mixes showed convincing deviation in pull out strengths for low performing strands like strand “C”. The mean of SCC-2 and SCC-3 proved that with more paste the mixes were almost identical in bond performance.

Concrete Type	Strand Type	Testing Date	STSB Tests Mean Concrete Strength (psi)	Pull Out Force at 0.1" Slip	STD Dev	COV (%)	w/c
C-N	A(0.5")	24-Jun-08	4259	14608	1116	7.64	0.46
SCC -1	A(0.5")	28-Jun-08	4368	12595	519	4.12	0.47
SCC -2	A (0.5")	4-Jul-08	4403	13864	486	3.50	0.47
SCC -3	A (0.5")	30-Jun-08	4192	12756	1170	9.17	0.45

Table 5.3: Summary of NASP Test Results in Strand “A” (0.5)

In contrary, for Strand “A” the coefficient of variation for the mixes were much

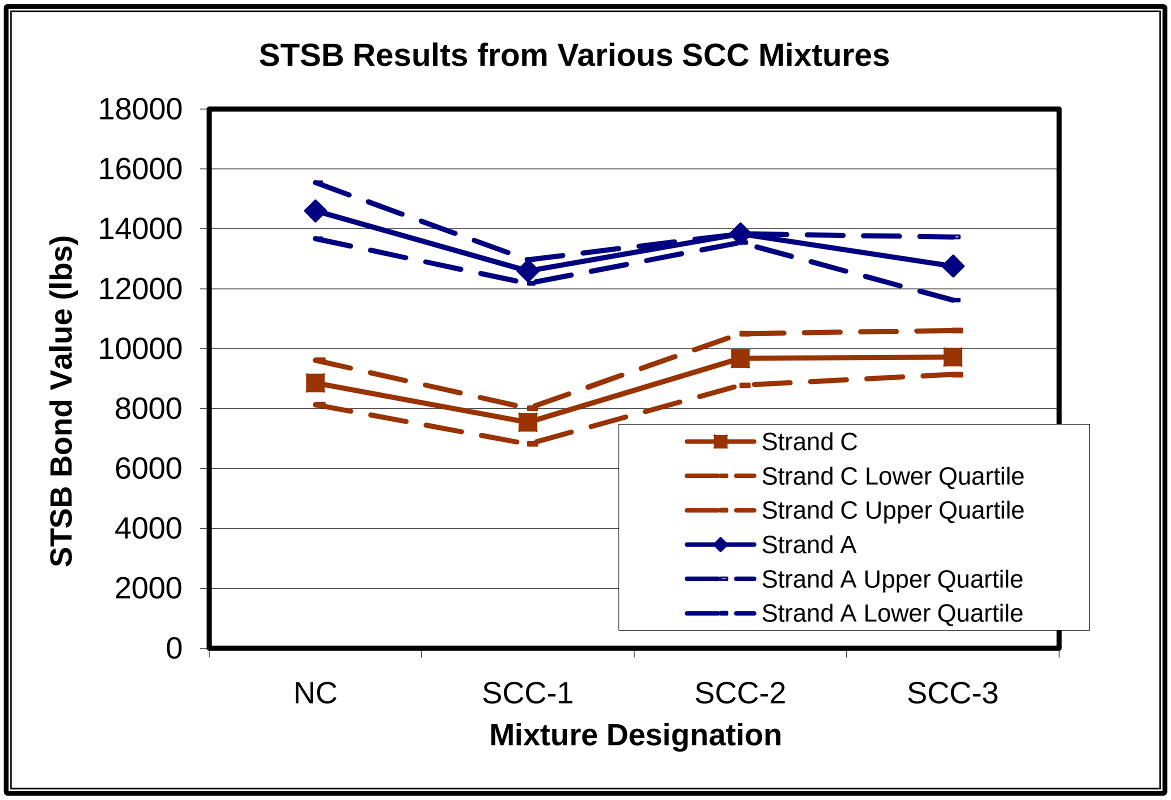


Figure 5.6: STSB Results showing the variation between NC and SCC.

Lower, than strand “C”. For strand “A”, the SCC-1 and SCC-2 showed the lowest variation in the pull out strengths. This means that these mixes behaved well with the high performing strands. The quartile plots plotted above gave the entire range of predicted bond values as well. In Strand, “A” the mixes SCC-1 and SCC-2 showed a narrowing range of the quartile plots indicating a smaller variation in the values as we proceed with these mixes, whereas for SCC-3 the deviation in the quartile range was increasing. The uppermost quartile for SCC-3 in strand “A” was almost equal to the lower quartile of the NC and. In contrary, for Strand “C”, the upper quartile points for SCC- 2, SCC- 3 are much higher than NC, showing higher bond strength.

CHAPTER VI

DISCUSSION OF TEST RESULTS AND CONCLUSIONS

Summary:

The research project aimed at finding the bond capacity of the SCC mixes. With the advent of high-performance concrete like SCC and its application in the pre-stressed industry, it is necessary to predict the bond performance of strands with this concrete. Thus NASP test was performed in order to evaluate the variability in the bond of strands with different SCC mixtures. The variables used are the two sources of 0.5 in strands and size, content of coarse aggregates. The fresh and hardened properties were determined from trial batching. Among nine batches, three SCC batches were finalized for bond tests. They met all the criteria's of fresh properties Slump Flow, J-Ring and L-Box tests. The hardened properties of the bond test as specified by its protocols that they should be in between 4400 to 5000 psi in 22-24 hrs. Thus, in order to account for the best SCC mixes, statistical computations are done in SAS. This gave us an idea about the nature of mixes which can be further modified to get desired bond pull out values. The test results are briefly discussed below:

6.1 Discussion of Test Results:

The SCC pull out values were analyzed for the set of concrete strengths. Then, the mixture which performed best with the lower performing strand and the one with higher performing strand was analyzed. In order to get a better comparison of the means, ANOVA test was performed. Then the normalized value of the NASP (SCC)/NASP (Mortar) was evaluated for both the strands and plotted against the concrete strengths.

Thus, the formulation derived from the graphs helped us in predicting the nature of the bond force of SCC to some extent. The set of analysis are tabulated as follows:

In order to start with, a confidence interval of 95% was chosen for the pull out loads at 0.1 in slip. Thus, it is important to go for general procedure of statistical tests it is mandatory to check the normality of the SCC mixes and the pull out values. As the sample size is less than 30, we can use the t-interval procedure for this test after it satisfies the normality distribution. Hence, the probability plots are plotted. “A normal probability plot is a plot of the sample data versus the data we would expect to get by taking a sample of the same size from a standard normal distribution. If the sample is from a normally distributed population, then the probability plot should be roughly linear. The inference drawn from the plot is subjective as we are using the sample to make a judgment about the population. (Weiss,1995). The normal probability plots for the SCC mix strengths and their corresponding pull out loads at 0.1in are plotted as follows:

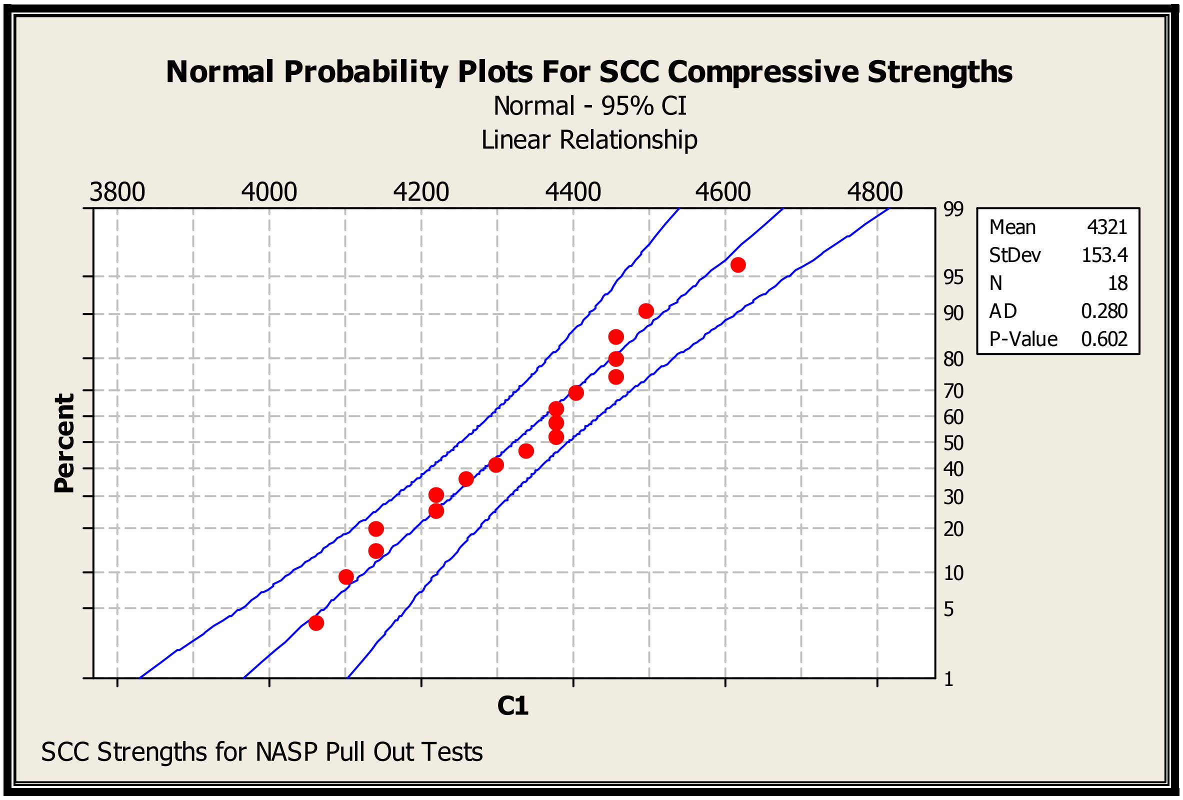


Figure 6.1: Probability Plots for SCC Strengths for all Mixes.

From the above graph we see that the normal probability plot, of the concrete strengths of 1 Day for SCC-1, SCC-2 and SCC-3 are roughly linear. Hence we subjectively conclude that its population is normally distributed. We also see that there are no visible outliers.

Now, we will check the normality of the NASP Pull out values for Strand “C” in SCC mixes.

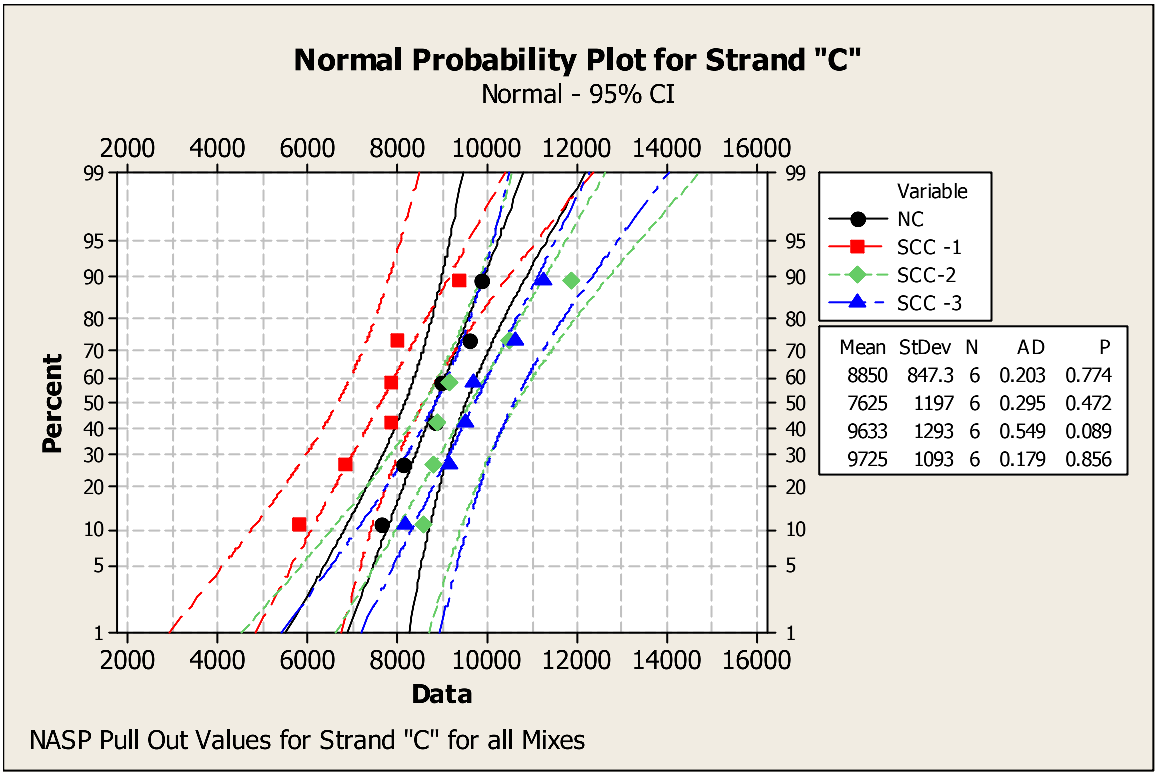


Figure 6.2: Probability Plots for Strand "C"

From the above graph we see that the normal probability plot, of the .1in pullout loads, with the SCC mixes are roughly linear. Hence we subjectively conclude that its population is normally distributed. We also see that there are no visible outliers. Thus, a little bit of skewness is ignored in the present scenario. Therefore, we can now proceed with other statistical analysis. The Normal Concrete Pull out values and the SCC pull out values showed very convincing results in the case of strand "C". The plots suggested that they were roughly linear. The normality probability plots for strand "C" showed that green dots (SCC-2) and blue dots (SCC-3) are very near to the NC bond values, or falling within the 95 % confidence interval, proving that the bond pull out at 0.1 in, for these mixes are nearly equal. However, SCC-1 was outside the C.I, but it was very closer to the

NC. Thus, we need larger sample size to reach in a final conclusion, which calls for further scope of research.

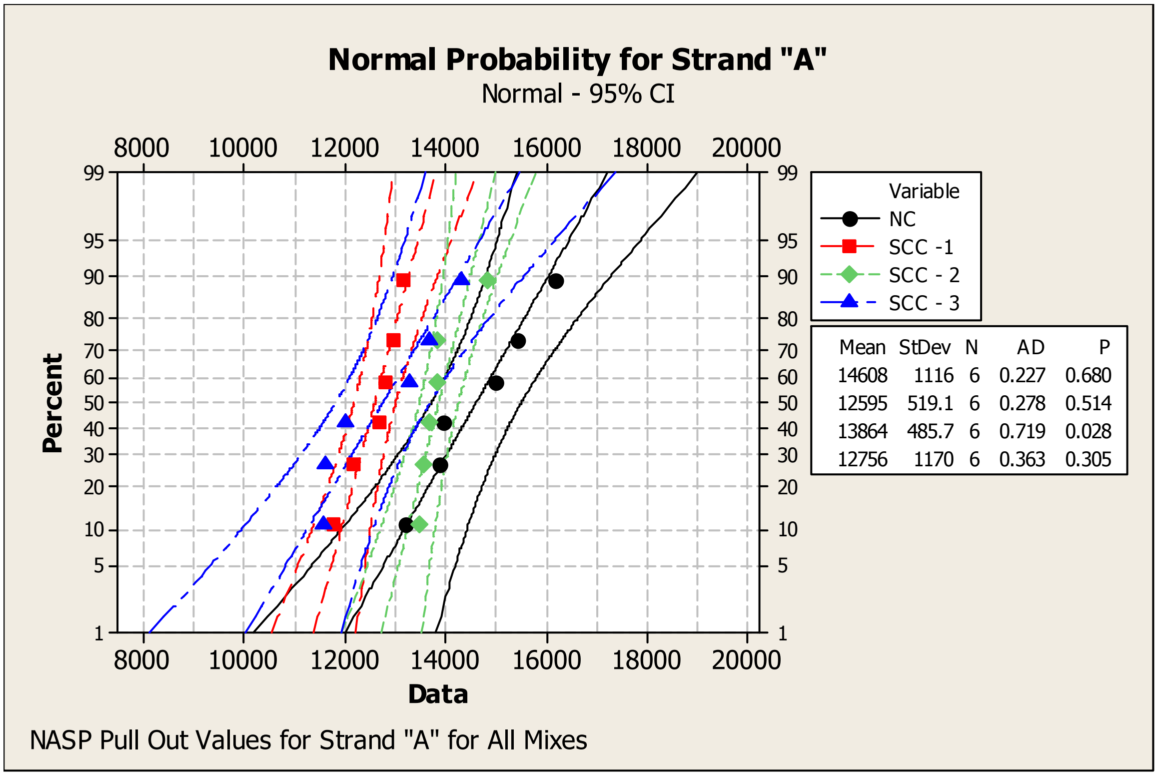


Figure 6.3: Probability Plots for Strand "A"

Thus, from the above plots we can say that the pull out loads are nearly normal with the SCC strength values for Strand "A". The population is very normal and there is no visible outliers in the data set. The plot above proved that there is a bit of skewness in the pull out values in the case of Strand "A". The variation in the bond strength of the strand "A" may be due to a variation in the methodology and process of testing. However, more quality control while testing can be adopted. From the C.I plot we can say that NC and SCC-2 have some similarity in the bond strength of the strands, however there was huge deviation in the pull out values for strand "A". The sample size was small enough to reach into a final conclusion for the deviation in the case of strand “A”, which calls for further scope of research in the bondability of SCC mixes.

In order to comapre the mean pull out strengths and to know the difference , the SCC mixes had on the bond strength we performed a multiple comparison test known as “Bonferroni test”. This test is valid on any equal and unequal sample sizes for any of the pairwise comparisons of the mixtures by this test.

From this results , we can conclude and predict which mix will behave better as compared to others with a particular type of Strand. The table below shows the total number of observations in which this test was done.

One-Way Analysis of Variance Results

The ANOVA Procedure

Class Level Information		
Class	Levels	Values
CONCRETE	4	NC SCC_1 SCC_2 SCC_3

Number of Observations Read	24
Number of Observations Used	24

Table 6.1: ANOVA Analysis on Strand “C”

As we have four mixes and each mixes were tested with six strands for NASP test , the total of all comes to be 24. The F value was tested with a confidence interval of 95%. The F-test showed that the variaces are equal among the groups and we can proceed with the multiple comparison t-tests, as this value is less than 0.05. The detailed analyis are shown as follows :

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	3	16990784.83	5663594.94	4.52	0.0142
Error	20	25084801.67	1254240.08		
Corrected Total	23	42075586.50			

R-Square	Coeff Var	Root MSE	PULLOUT_FORCE Mean
0.403816	12.50164	1119.929	8958.250

Source	DF	Anova SS	Mean Square	F Value	Pr > F
CONCRETE	3	16990784.83	5663594.94	4.52	0.0142

Table 6.2: Summary of ANOVA Test for Strand "C".

The Bon-Ferroni test showed below indicated below means that the Type II error is a bit higher than Type I. The Type II error occurs when we don't reject the null hypothesis when it is in fact false. Our assumption, is for the null hypothesis that all the means are the same and alternate hypothesis that at least one mean is different. The chances of having a Type II error means that we fail to reject the null hypothesis when it is false. In other words , we say we accept that our null hypothesis of having equal means is true, however when it is incorrect. Thus, in our case we conclude that there is a difference in the means of the pull out test for Strand "C" with the SCC mixtures. In order to correlate and find the relationship between the SCC mixes, we perfomed the ANOVA test of multiple comparisons known as "BonFerroni Test". From this test we can conclude which mixtures are close enough and which will give a better bond as compared to another. This also performs comparisons among the group and in between the group.

The table below is a summary of the test given. The test is carried at a significance level of 0.05. The purpose of t-test was to evaluate the mean difference in the group variables. The critical value obtained proved that we are rejecting the null hypothesis of having equal bond strength of the mixes. Thus, for strand "C" there is a significant difference in the pull out values between the groups which is close to 1890 lbs

Bonferroni (Dunn) t Tests for PULLOUT_FORCE

Note: This test controls the Type I experimentwise error rate, but it generally has a higher Type II error rate than REGWQ.

Alpha	0.05
Error Degrees of Freedom	20
Error Mean Square	1254240
Critical Value of t	2.92712
Minimum Significant Difference	1892.6

Table 6.3: Bon-ferroni Test on Strand "C"

However, the summary statistic for the Bon-Ferroni test is shown below. This can be said that the entire test is divided into major groups "A" and "B". The grouping showed the mean and the number of samples tested in each case. The SCC-3, SCC- 2 and NC showed no significant difference in the pull out values. This means that for low performing strands like Strand "C", the mix design of SCC had hardly any effect in the bond strength and pull out values. This means that with all the other factors like w/c ratio, the fine aggregate content and admixture dosage rates keeping constant the factor of reducing the coarse aggregate content and size of SCC's as compared to the normal concrete makes no significant difference in the pull out values.

Means with the same letter are not significantly different.				
Bon Grouping		Mean	N	CONCRETE
	A	9725.0	6	SCC_3
	A			
	A	9632.8	6	SCC_2
	A			
B	A	8850.0	6	NC
B				
B		7625.2	6	SCC_1

Table 6.4: Results of Bon-Ferroni Test on Strand "C"

On the contrary, for NC and SCC-1, there is a significant difference in the pull out values at 0.1" slip. This can be interrelated that the SCC behaves more likely the NC with more reduction of coarse aggregate than only a small amount. The SCC mixes behaved much better with more of fines. Also from the fresh properties, we concluded that if more fines and well graded particles are used in making SCC we can have a better mix with respect to segregation ability, passing and filling ability.

However, by varying the ingredients in the mixture calls for further scope of research in SCC bond test. Lastly, a descriptive summary is given in the next page as follows. The table below showed the descriptive summary of all the concrete mixes.

CONCRETE	Mean of PULLOUT_FORCE	Std. Dev. of PULLOUT_FORCE	Std. Error of PULLOUT_FORCE	Variance of PULLOUT_FORCE	Number of non-missing values for PULLOUT_FORCE	Number of missing values for PULLOUT_FORCE	Minimum of PULLOUT_FORCE	Maximum of PULLOUT_FORCE
	8958.25	1352.54	276.087	1829373.33	24	0	5830	11867
NC	8850.00	847.34	345.924	717981.20	6	0	7643	9856
SCC_1	7625.17	1196.98	488.664	1432754.97	6	0	5830	9367
SCC_2	9632.83	1292.63	527.714	1670894.17	6	0	8569	11867
SCC_3	9725.00	1093.31	446.343	1195330.00	6	0	8160	11250

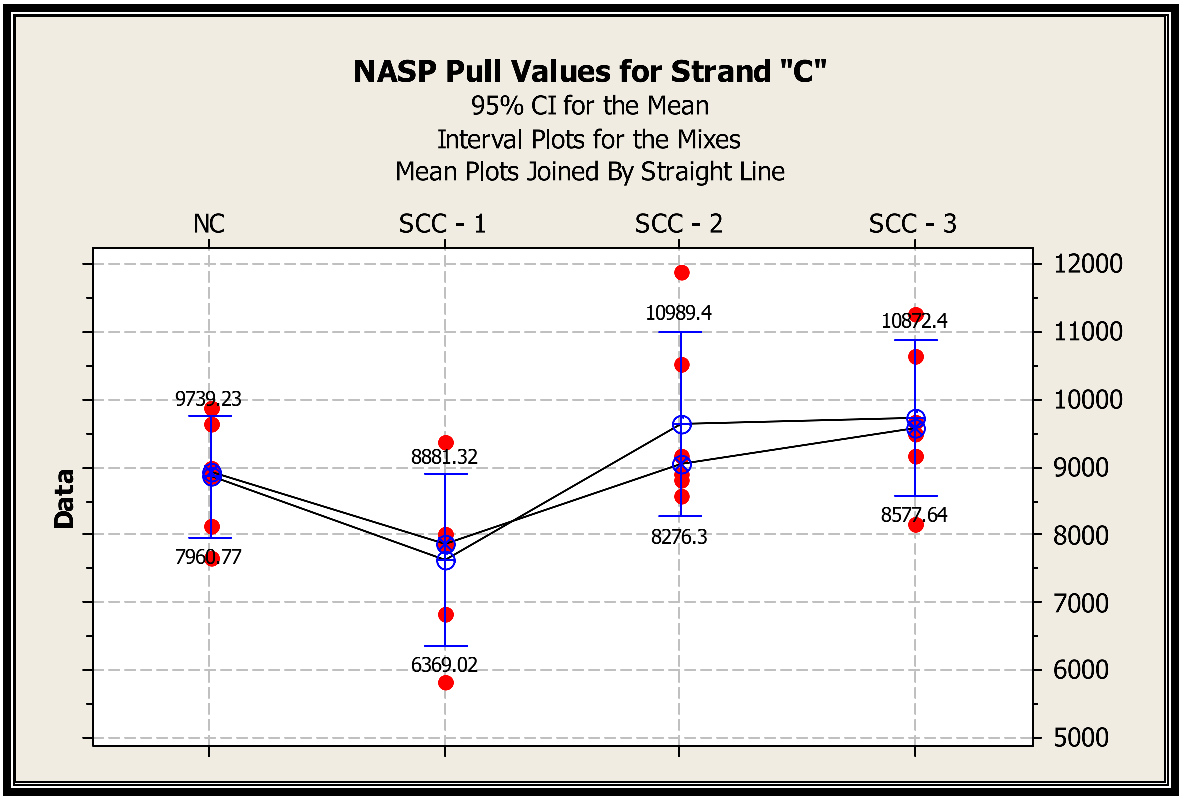


Table 6.5: Means by Box- Whisker Plot for Strand "C"

The above table is an interval plot for each single mixes showing the lowest, highest and the means. The individual data points are shown in red dots. The outliers are the points that are outside the interval ends in each bar. The means and the medians of the mixes are almost in the same levels and equal for each. It depicts that low performing strands like Strand "C", the 0.1" slip values are more for SCC than the normal concrete. It can be said that irrespective of the changes in the mix design, the bond strength for SCC-2 and SCC-3 are almost similar for Strand "C", though there is substantial difference with SCC-1 and NC. This recommends for further research with SCC mixes and bond of pre-stressing strands with respect to more fines, well graded rocks and usage of reduced rock size to achieve the desired degree of pull out values.

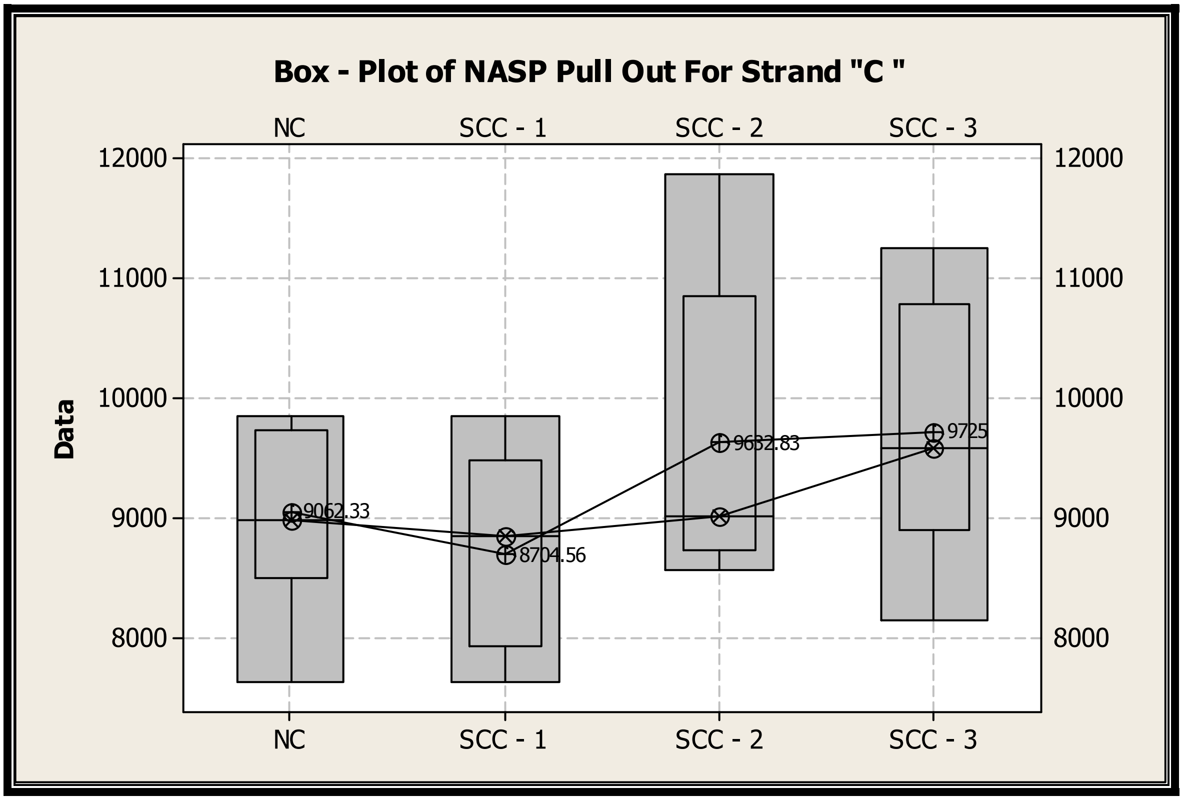


Table 6.6: Box-Plot for Strand “C”

The above plot shows the schematic representation of the box plot of the pull out values of the SCC and NC mix. The bigger box is the range box, the smaller one is the inter-quartile range box. The inter-quartile range is the difference between the first and the third quartiles. The lines connecting the means and the median are the median connecting line which shows the mean difference in the pull out values of the mix. The quartile divides the data set into four quarters or four equal parts. The first quartile divides the bottom 25% of the data from the top 75%, second quartile divides the bottom 50% of the data from the top 50% and the third quartile divides the bottom 75% from the top 25%. On the above diagrams the two lines emanating from the box are called the whiskers. The first quartile is called the lower hinge and the third quartile is called the upper hinge. The essence of having the box plot is that it gives an idea of the skewness in the data set.

One-Way Analysis of Variance Results

The ANOVA Procedure

Class Level Information		
Class	Levels	Values
Concrete	4	NC SCC_1 SCC_2 SCC_3

Number of Observations Read	24
Number of Observations Used	24

Table 6.7: ANOVA Analysis for Strand "A"

Similarly, the variance analysis was also carried on Strand "A". The table above shows the number of samples tested for bond in all. The type of concrete was four, likely NC, SCC-1, SCC-2 and SCC-3.

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	3	16352542.46	5450847.49	6.99	0.0021
Error	20	15600377.50	780018.88		
Corrected Total	23	31952919.96			

R-Square	Coeff Var	Root MSE	Pullout_Force Mean
0.511770	6.563740	883.1868	13455.54

Source	DF	Anova SS	Mean Square	F Value	Pr > F
Concrete	3	16352542.46	5450847.49	6.99	0.0021

Table 6.8: Summary Analysis for Strand "A"

The statistical analysis shown above was carried at a significance level of 5%. The probability of the F-Value is less than required which means that the mean pull out values of the strand is not same. In order to get the exact analysis of the comparison, we performed the Bon-Feroni test of multiple comparison for single variable like Strand "A".

Bonferroni (Dunn) t Tests for Pullout_Force

Note: This test controls the Type I experimentwise error rate, but it generally has a higher Type II error rate than REGWQ.

Alpha	0.05
Error Degrees of Freedom	20
Error Mean Square	780018.9
Critical Value of t	2.92712
Minimum Significant Difference	1492.6

Table 6.9: Bon-Ferroni Test for Strand "A"

The table above shows the critical value for the t-test after analysis and the corresponding significant difference. The significance difference in the pull out values was found to be 1492.6 lbs. In order to get the better correlation between the mixes and the pull out values the Bon-Ferroni test was carried out.

Means with the same letter are not significantly different.				
Bon Grouping		Mean	N	Concrete
	A	14608.0	6	NC
	A			
B	A	13863.7	6	SCC_2
B				
B		12755.8	6	SCC_3
B				
B		12594.7	6	SCC_1

Table 6.10: Results of Bon-Ferroni Test on Strand "A"

The table above showed that there was no major difference between NC and SCC-2 in terms of the mean pull out values, however there is a significant difference between SCC-

3, SCC-1 against NC. This can be interpreted as on reducing the content of coarse aggregate and content we will have a better SCC in terms of fresh properties, but there is a significant difference in the pull out values. This means that the normal concrete gives better bond or as comparable to SCC with more reduction of rock size. However, due to the variability in the mix designs and better approach to make SCC similar or higher bonding capacity can be achieved. This calls for further scope of research.

The table below shows the summary statistics of Strand "A" and it's bond performance.

One-Way Analysis of Variance
Results
Means and Descriptive Statistics

Concrete	Mean of Pullout_Force	Std. Dev. of Pullout_Force	Std. Error of Pullout_Force	Variance of Pullout_Force	Number of non-missing values for Pullout_Force	Number of missing values for Pullout_Force	Minimum of Pullout_Force	Maximum of Pullout_Force
	13455.54	1178.67	240.595	1389257.39	24	0	11577	16184
NC	14608.00	1116.41	455.771	1246361.20	6	0	13214	16184
SCC_1	12594.67	519.11	211.925	269472.27	6	0	11781	13153
SCC_2	13863.67	485.72	198.295	235926.27	6	0	13495	14813
SCC_3	12755.83	1169.75	477.549	1368315.77	6	0	11577	14320

Table 6.11: Summary Statistics for Strand "A

The mean pull out force was 13455 lbs for the entire set. In all the mixes, the standard deviations of the pull out values were below 1500 lbs, which satisfies the test criteria. The maximum and minimum values for each single set were shown against each mixes. SCC-1 and SCC-2 behaved much better as the standard deviations were less and more close to each than SCC-3 and NC-1. That means, for high strength strands the lesser coarse aggregate content works well than reduced rock size.

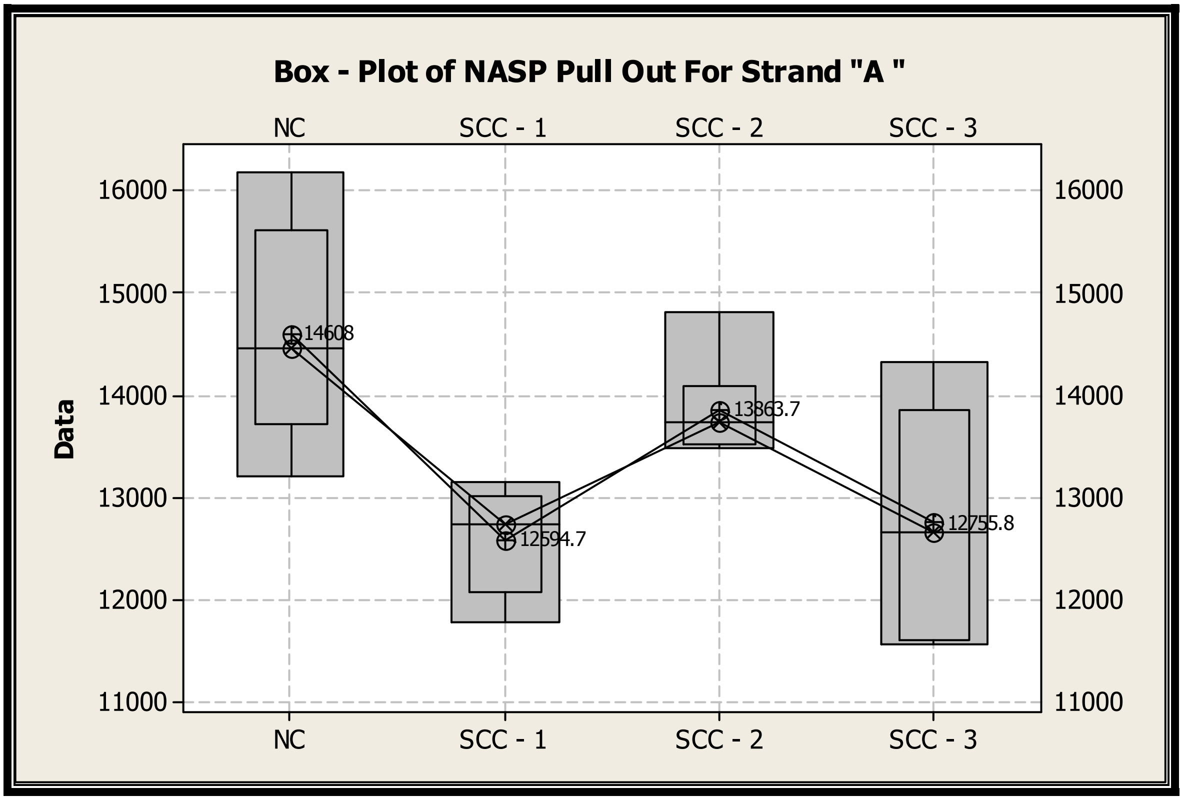


Table 6.12: Box-Plot for Strand “A”

The above plot shows the data range, inter-quartile range, means, medians and the whiskers for the strand “A” pull out values for the mixes. The mean of NC was the maximum , SCC-2 was higher than the other two mixes. Also, from the Bon-Ferroni test we found the same conclusion. The SCC-1 and SCC-3 slip values are almost in the same range. The NC and SCC-3 values are normal as the mean and the median are almost in the centre of the box. The SCC-1 is right skewed and SCC-2 is left skewed, that means the values are not normally distributed about the means. However, this does not reflect the nature of the mix and adds to further scope of research in SCC and its ability to bond with pre-stressing strands.

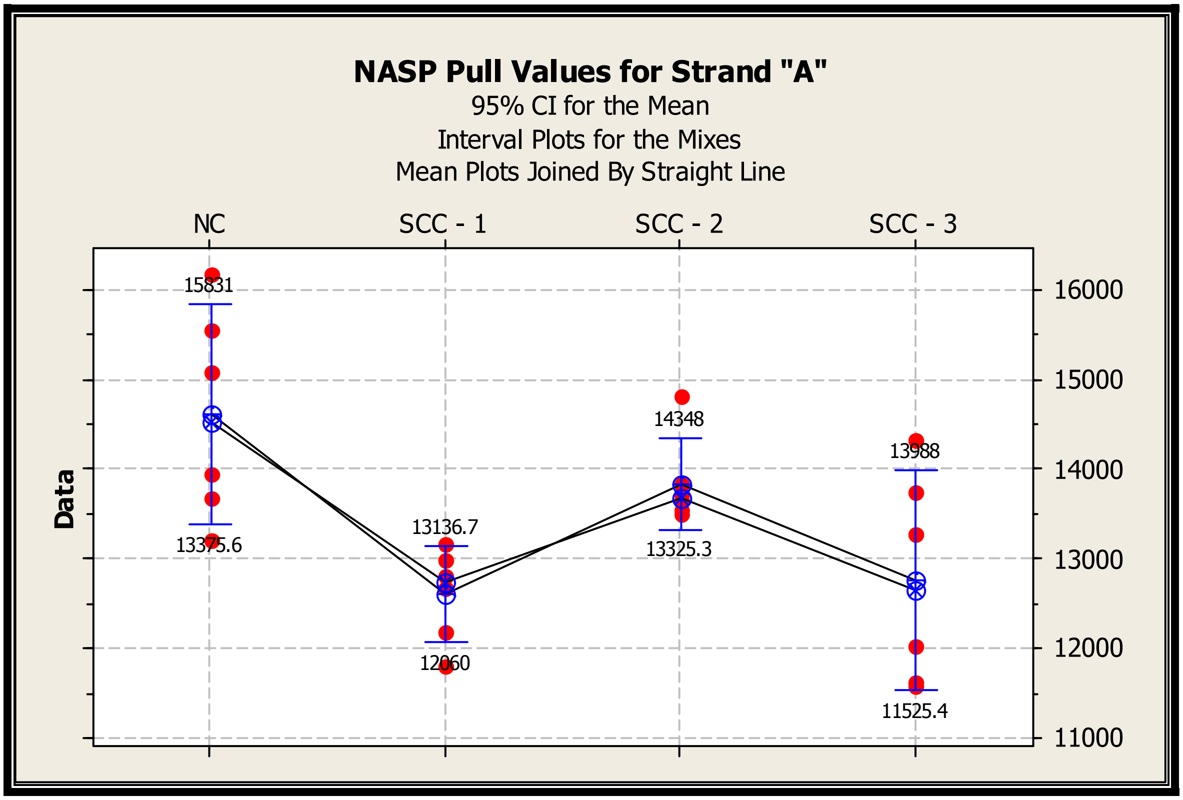


Table 6.13: Means of Box-Plot for Strand "A"

The above plot is an interval plot for the strand "A" which shows the nature of distribution of the individual plots in both sides of the means. The outlines are the values outside the end intervals in the respective bands. The means and the medians almost match for all the mixes. The mean for SCC- 3 was the lowest of all, and the individual pull out values is not closer to the mean as well. The SCC-2 was a better mix in strand "A" than SCC- 1 and SCC-3. That means, for less coarse aggregate content SCC may behave very closer to the normal concrete if the other parameters are kept same like gradation, stringent moisture control and admixture dosage rates. Thus, in SCC-2 only outliner was found and the other values are very closer to the mean. The decrease in rock size had a negative impact on bond pull out values of strand " A", as all the values are having a wider range than others. Thus, further research can be done on strand A with various other parameters of SCC.

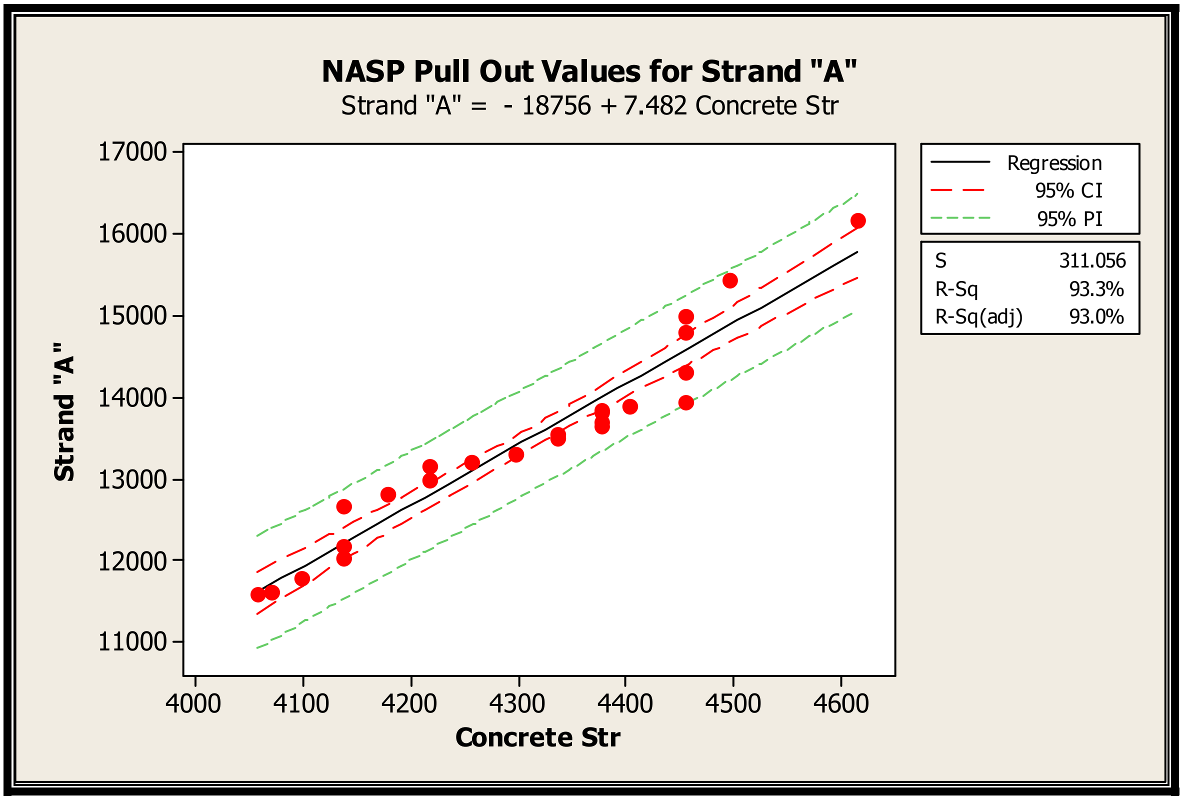


Table 6.14: Regression of Strand “A” with Concrete Strength

Thus, in order to establish a relationship between the pull out values and SCC strength a linear fitted line plot was plotted. The plot showed a linear relationship between the bond strength and SCC strength with a coefficient of regression about 0.93. Most of the data set lies within the 95% of the predicted interval but outside the confidence interval. That means if we need high strength SCC, we can get higher pull out and bond strength as well. The straight line equation also depicts the linear relationship among the mixture types and variables used in the SCC mixes. Though there was a huge variation in the fresh and rheological properties in the SCC mixes, it still showed the same bond properties within a certain range of given w/c ratios.

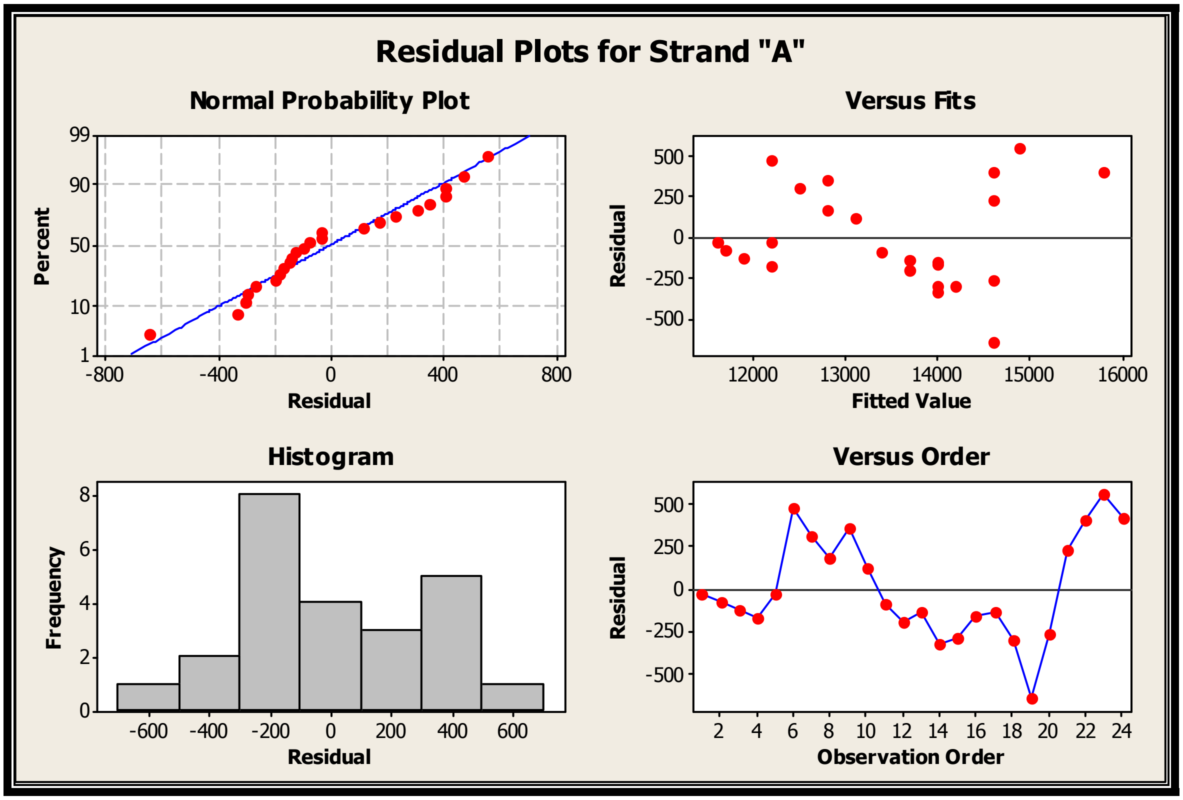


Table 6.15: Residual Plots for NASP Pull Out for Strand “A”.

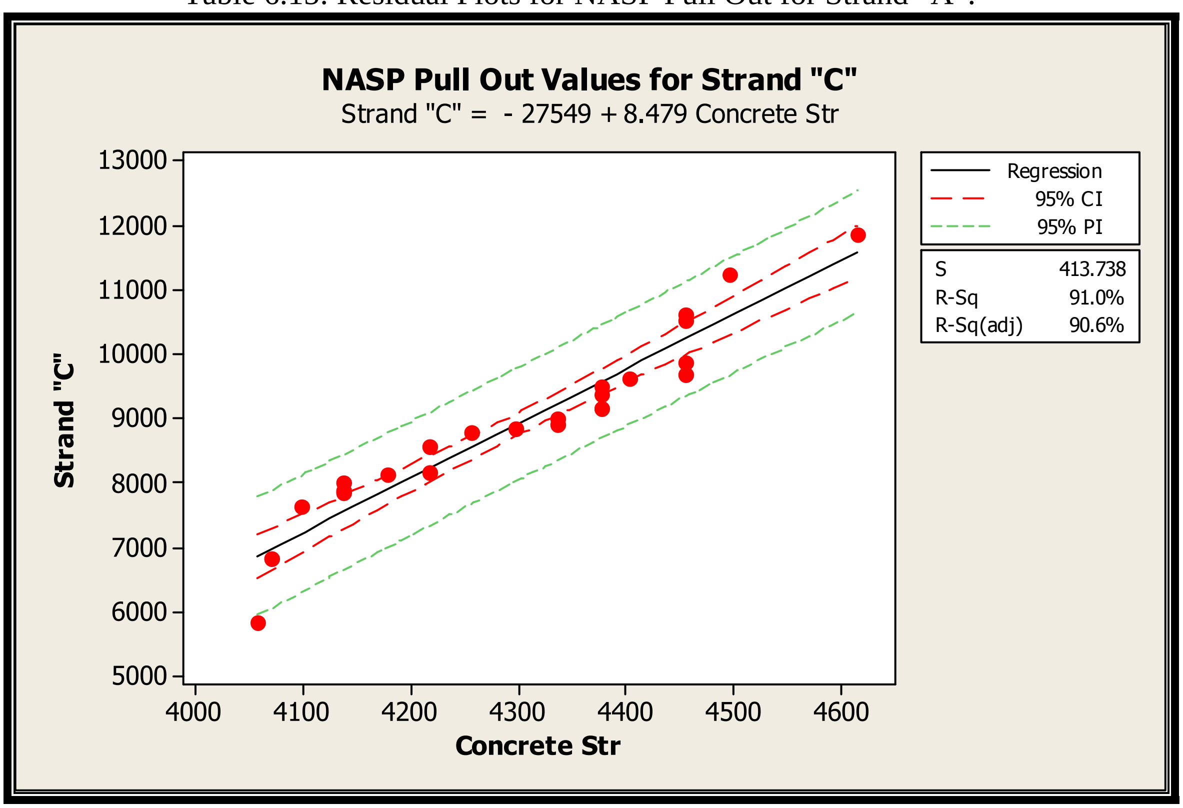


Table 6.16: Regression of Strand “C” with Concrete Strength

The Table 6.16 showed the regression line with the fitted plot for Strand “C” against the concrete strength. The plot was almost linear within the strength range of 4000 to 4600 psi concrete strength. The coefficient of regression for strand “C” was 0.91. Around half of the data points are within the confidence interval of 95 % , but there was wider range observed when we analyze the pull out strengths taking into consideration of the NC and SCC mixes.

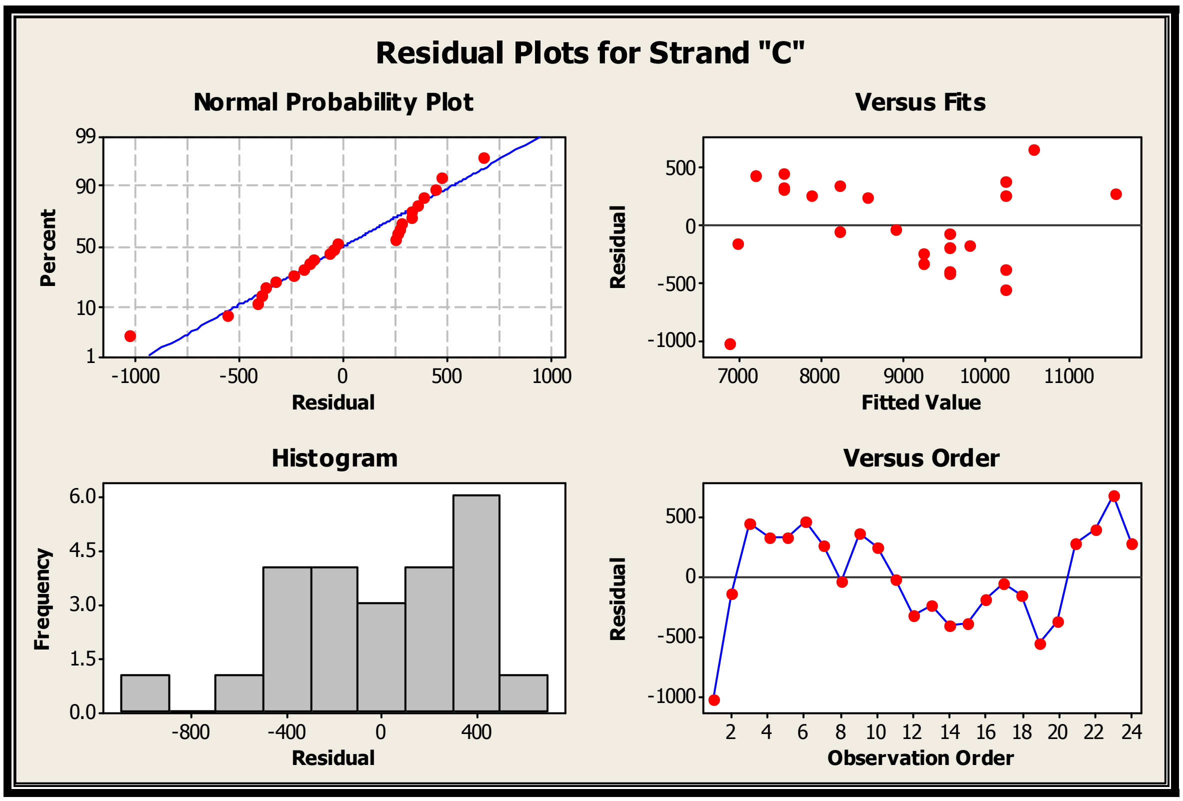


Table 6.17: Residual Plot for NASP Pull Out for Strand “C”

Comparison of NASP SCC Results with NASP Mortar Results:

The NASP mortar results were higher than the pull out values tested against the Self consolidating mixes. The mortar grout provided more friction and mechanical interlocking in the surface of the strands than the concrete. This may be due to the ability of the grout to adhere more into the helical grooves of the wires, resisting the further slipping of the king wire by mechanical interlocking. The content of coarse aggregate had hardly any effect in the bond strength of the pre-stressing strands. However, different gradations of coarse aggregate from various sources, could have been a further scope of research in this field of study ahead. There was a difference observed in the pull out values in the Normal Concrete Mix tested by Eden (2006) against the strand “A”, as compared to the mixes tested in this research. This may due to some variation in the testing methodology which had happened unknowingly. However, the SCC bond values, was similar to Normal Concrete in both the strands “A” and “C”, proving that it can be successfully used in the concrete industry by reaping its benefits.

Conclusions and Recommendations:

The SCC trial batching was done with only one variable which was the coarse aggregate size and content. The SCC-1, SCC-2 and SCC-3 were all having different slump flows. L-Box , J-ring values though the dosages of HRWRA, VMA and Recover were kept constant throughout. The statistical analysis showed that the NASP test is effective in finding the bond properties with varying the concrete properties. The NASP bond forces at 0.01 and 0.1 in free end slip varies linearly with the compressive strengths of the concrete or the mortar. The compressive strength is directly proportional to the NASP bond force.

The NASP Bond test is a repeatable and reliable test method.

The NASP Bond test can be performed on 0.5 in and 0.6 in diameter Grade 270 pre-stressing strands.

The bond value increases both with increasing mortar and concrete strength.

The NASP test can be accepted as a standardized test for strand bond as it had showed higher correlations than Moustafa and PTI bond test (Chandran, 2004).

The SCC mixes can be altered as per project specifications and need of the user. In order to achieve high strength SCC, lower w/c ratios can be selected. However, if we reduce the w/c ratio below a certain extent, it’s important to prorate the admixture dosages accordingly in order to attain the flow-abilty, segregation resistance , filling and passing ability.

It was found that on reducing better well graded rock and less size the better SCC can be developed. The fines in the mix are the key to have successful mixture properties in SCC. Though the admixture used are a bit expensive, a cost effective SCC can also be tried in the bond test by increasing the powder, paste and binder content. This hardly will have any effect in the bond strength.

The SCC and Normal Concrete pull out values were almost the same in the case of Strand “C” but there was a difference observed in strand “A” which calls for further scope of research in this field. This may be due to the variation in the testing methodology.

REFERENCES

ACI Committee (2005)“Building Code Requirements for Structural Concrete (ACI 318 -02) and Commentary (ACI 318R-05)”.ACI, Farmington Hills, Michigan.

Anderson, A.R and Anderson, R.G (1976) “An Assurance Criterion for Flexural Bond in Pre-tensioned Hollow Core Units.” *ACI Journal*, 72(8),457-464.

Barnes. R.W., Grove, J.W., Burns,N.H.(2003). “Experimental assessment of factors affecting transfer length.” ACI Structural Journal,100 (6).740-748.

Brown, M.D. (2003). “Evaluation of Test Methods in Assessing Bond Quality of Pre-stressing Strand ”. University of Oklahoma, Norman.

Cousins,T., Johnston ,D., and Zia, P.(1990). “ Transfer and Development Length of Epoxy Coated Pre-stressing Strand.” *PCI Journal*,37(1),66-73.

Ferzli, Y.(2000). “Validation of acceptance criteria for the bond of pre-stressing strands”. University of Oklahoma, Norman.

Grieve,C.B.(2004). “ Investigating the effect of mortar strength and loading rate on the NASP bond Test.” Oklahoma State University, Stillwater.

Janney, J.R. (1954). “Nature of Bond in Pre-tensioned Prestressed Concrete.” Journal of American Concrete Institute,50 (9),717-736.

Logan, D.R. (1997). “Acceptance Criteria for Bond Quality of Strand for Pretensioned Pre-stressed Concrete Applications.” PCI Journal, 42 (2),52-90.

Mitchell, D. Cook, W.D. Khan, A. A., Tham, T.(1993). “Influence of High Strength Concrete on Transfer and Development Length of Pre-tensioning Strand.” PCI Journal, 38(3), 52-66.

Portland Cement Association, PCA Design and Control of Concrete Mixtures,14th Edition, Engineering Bulletin 001.

Paulsgrove, G.A., R.W.B. (1999). “NASP Strand Bond Testing, Round Two,Assessing Repeatability and Reproducibilty of the Moustafa Test, the PTI Test, and the NASP Bond Test.” University of Oklahoma, Norman.

Rose,D.R,. Russell, B.W (1997). “Investigation of Standardised tests to measure the bond performance of prestressing strand.”PCI Journal, 42 (4), 56-80.

Russell, B.W (1992). “ Design Guidelines for Transfer, Development and Debonding of Large Diameter Seven Wire Strands in Pre-tensioned Concrete Girders.” University of Texas, Austin.

Tabatai,H.D., T.J., (1993). “ The History of the Prestressing Strand Developemnt Length Equation.” Construction Technology Laboratories, Virgiania.

Tessema, E.(2006). “ The Effect of High Strength Concrete on the Bondability of Pre-stressing Strands”.

Aaron W. Saak., Hamlin M. Jennings, Surendra P. Shah. (2001) “ New Methodology for Designing Self-consolidating Concrete”, ACI Materials Journal , Nov- Dec.,431-439.

Okamura H., Ouchi M., “ Self-Compacting Concrete Development, Present Use and Future.” Proc, 1st Inter RILEM Symposium on SCC, Sweden, Proc 7, 1999,pp 3-14.

Van K. Bui, Y. Akkaya and Surendra P.Shah.(2002) “ Rheological Model For Self-Consolidating Concrete”, ACI Materials Journal, Nov-Dec, 99 (6), 549-559.

Ulf Skarp, Jens Engstrand, Inger Jansson , “ A concept of Enhancing Early Strength Development In Self-Consolidating and Normal Concrete By Means of Increasing Stability and Homogenity”. pp- 363- 369.

Celik Ozylidrium , D.Stephen Lane, (2003), “ Final Report on Evaluation of Self-Consolidating Concrete.” June, pp. 1-14.

Khayat , K.H., C.Hu, H.Monty, (1999), Stability of Self-Consolidating Concrete, Advantages, and Potential Applications. In *Self Compacting Concrete: Proceedings of the First International RILEM Symposium,* A. Skarendahl, and O.Petersson, Eds, RILEM Publications, Cachen Cedex, France, 1999, pp – 143-152.

M. Lachemi , Khandaker M.A, Hossain,. V.Lambros, Nabil Bouzoubaa, (2003), “ Development of Cost-Effective Self-Consolidating Concrete Incorporating Fly Ash, Slag Cement or Viscosity Modifying Admixtures”. ACI Materials Journal, Sept-Oct, 100(5), 419 -425.

E.K. Attiogbe., Heather T.See, Joseph A. Daczko.,(2002)., “Engineering Properties of Self-Consolidating Concrete”; First North American Conference on the Design and Use of Self-Consolidating Concrete, Hanley Wood Publications, Nov-Dec, 331-336.

David Bonen, Surendra P Shah, (2005)., “Fresh and Hardened Properties of Self-Consolidating Concrete”, Concrete Construction, Dec 7, 14-26.

Hassan EI-Chabib , Moncef Nehdi, (2006)., “Effect of Mixture Design Parameters on Segregation of Self-Consolidating Concrete”., ACI Materials Journal, Sept-Oct , 103 M 42, 374 – 383.

M. Sonebi, Steffen Grunewald, Joost Walraven, (2007)., “Filling Ability and Passing Ability of Self-Consolidating Concrete”, ACI Materials Journal, March-April, 104-M19,162- 170.

R.P.Ferron., Amedeo Gregori, Zhihiu Sun, Surendra P.Shah (2007), "Rheological Method to Evaluate Structural Build –Up in Self-Consolidating Concrete Cement Pastes." ACI Materials Journal, May-June, 104- M26, 242-250.

Vasilios Bill Lambros (2003), "Self-Consolidating Concrete: Rheology, Fresh Properties and Structural Behaviour". Dec, 120-130.

Brian Mitchell (2004), "Strength and Material Properties of Self-Consolidating Concrete (SCC)",

Anton K. Schindler, Robert W. Barnes, J.B. Roberts, S.Rodriguez (2007). "Properties of Self- Consolidating Concrete for Pre-stressed Members". ACI Materials Journal, Jan – Feb 104- M07, 53- 61.

Introductory Statistics, By Neil.K. Weiss, 4th Edition, 573-621.

Annual Book of ASTM Standards, (2005), Section 4, Contruction.

Joseph, J. Assaad, Kamal H. Khayat, (2006), "Effect of Viscosity-Enhancing Amixtures on Formwork Pressure and Thixotropy of Self- Consolidating Concrete." ACI Materials Journal , July –Aug, 280 -287.

Kamal H. Khayat, Joseph J. Assaad, (2006), "Effect of w/cm and High Range Water Reducing Admixture on Formwork Pressure and Thixotropy of Self-Consolidating Concrete", ACI Materials Journal, May – June, 103- M21, 186 – 193.

Soo -Duck Hwang, Kamal H. Khayat, Olivier Bonneau (2006). " Performance –Based Specifications of Self-Consolidating Concrete Used In Structural Applications", ACI Materials Journal, March – April, 103 – M 14, 121 – 129.

PCI Interim Guidelines for the Design and Use of Precast and Pre-stressed Members.

APPENDIX A

11-2006 FRI 11:23 AM MCCORMICK WAREHOUSE FAX NO. 405 943 1117 P. 01

4-11-06 10:46A Hand1 Sak 405 789 3008 P.02

BUZZI UNICEM USA

PO Box 482-Greencastle, IN 46135-(765) 653-9766

This is to certify that Type III meets ASTM C-150 Specifications for Portland Cement.

Chemical Data ASTM C114		Physical Data ASTM C185	
Silicon Dioxide (SiO_2)	20.46	Air Entrained (%)	7.0
Aluminum Oxide (Al_2O_3)	5.38	ASTM C204	
Ferric Oxide (Fe_2O_3)	2.39	Fineness (cm^2/gm)	5960
Calcium Oxide (CaO)	63.05	ASTM C151	
Magnesium Oxide (MgO)	1.53	Autoclave Expansion (%)	-0.010
Sulfur Trioxide (SO_3)	4.18	Compressive Strength, PSI	
Loss on Ignition	2.14	ASTM C109 Mortar Cubes	
Sodium Oxide	0.12	1-Day	4120
Potassium Oxide	0.90	3-Day	5560
Insoluble Residue	0.70	7-Day	6470
Total Alkali as Na_2O	0.72	28-Day	
POTENTIAL COMPOUND COMPOSITION		ASTM C191	
		Setting Time:	
		Vicat	
Tricalcium Silicate (C_3S)	50	Initial, Min.	72
Dicalcium Silicate (C_2S)	21	Final, Min.	143
Tricalcium Aluminate (C_3A)	10		
Tetracalcium Aluminoferrite(C_4AF)	7		

Silo	Bill of Lading	Tons	Date	Silo	Bill of Lading	Tons	Date
				701			8/6/2006

STATE OF INDIANA)
COUNTY OF PUTNAM)

Before me the undersigned, a Notary Public for Putnam County, State of Indiana personally appeared John J. Wachal and acknowledged the execution of the foregoing instrument this 9th day of August 2006

Attn:
Fax.

John J. Wachal

John J. Wachal
Quality Manager

Philip A. Cledfelter, Notary Public
My commission expires May 13, 2008.

AUG-11-2006 FRI 11:13 AM MCCORMICK WAREHOUSE FAX NO. 405 943 1117 P. 01

P. 01

TRANSACTION REPORT

AUG-10-2006 THU 03:37 PM

DATE	START	RECEIVER	TX TIME	PAGES	TYPE	NOTE	M#	DP
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				TOTAL :		0S PAGES: 0		

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BUZZI UNICEM USA

PO Box 482-Greencastle, IN 46135-(765) 653-9766

This is to certify that Type III meets ASTM C-150 Specifications for Portland Cement.

Chemical Data		Physical Data	
ASTM C114		ASTM C185	
Silicon Dioxide (SiO_2)	20.45	Air Entrained (%)	7.0
Aluminum Oxide (Al_2O_3)	5.38	ASTM C204	
Ferric Oxide (Fe_2O_3)	2.39	Fineness (cm^2/gm)	5960
Calcium Oxide (CaO)	63.05	ASTM C151	
Magnesium Oxide (MgO)	1.53	Autoclave Expansion (%)	-0.010
Sulfur Trioxide (SO_3)	4.10	Compressive Strength, PSI	
Loss on Ignition	2.14	ASTM C109 Mortar Cubes	
Sodium Oxide	0.12	1-Day	4120
Potassium Oxide	0.90	3-Day	5560
Insoluble Residue	0.70	7-Day	6470
Total Alkali as Na_2O	0.72	28-Day	
POTENTIAL COMPOUND COMPOSITION		ASTM C191	
		Setting Time:	
Tricalcium Silicate (C_3S)	50	Vicat	
Dicalcium Silicate (C_2S)	21	Initial, Min.	72
Tricalcium Aluminate (C_3A)	10	Final, Min.	143
Tetracalcium Aluminoferrite (C_4AF)	7		

APPENDIX B

W. R. GRACE
MATERIAL SAFETY DATA SHEET

Product Name: ADVA Cast 575
MSDS ID Number: D-06201 MSDS Date: 11/14/2006

SECTION 1 - CHEMICAL PRODUCT AND COMPANY IDENTIFICATION

Product Name: ADVA Cast 575
MSDS Number: D-06201
Cancelled MSDS Number: D-06195
MSDS Date: 11/14/2006
Chemical Family Name: Carboxylated Polyether
Product Use: Concrete Additive
Chemical Formula: Mixture-NA
CAS # (Chemical Abstracts Service Number): Mixture-NA

Manufactured by:

W.R.Grace & Co.-Conn.
62 Whittemore Avenue
Cambridge, MA 02140

Grace Canada, Inc.
294 Clements Road West
Ajax, Ontario L1S 3C6

In Case of Emergency Call:
In USA: (617) 876-1400 In Canada: (905) 683-8561

SECTION 2 - COMPOSITION/INFORMATION ON INGREDIENTS

Ingredient	CAS#	Percent (max)
Carboxylated polyether copolymer	Proprietary	10-20

SECTION 3 - HAZARDS IDENTIFICATION

Emergency Overview:
Caution!
May cause skin and eye irritation

HMIS Rating:
Health: 1
Flammability: 1
Reactivity: 0
Personal Protective Equipment: B (See Section 8)

Potential Health Effects:

Inhalation: Acute inhalation not expected to result in adverse effects.
If inhaled as a vapor or mist, causes irritation, sore throat, coughing and breathing difficulty.
Effects include: Nausea, vomiting, headache, tightness of chest, hypersensitive individuals may experience allergic respiratory reaction and wheezing.
Eye Contact: Eye contact may cause irritation.
Skin Contact: Skin contact may cause irritation.
May cause sensitization.
Hypersensitive individuals may develop an allergic reaction resulting in dermatitis, rash or hives.
Skin Absorption: Not expected to be harmful if absorbed through the skin.
Ingestion: Ingestion not expected to be harmful.
If ingested, causes irritation to the linings of the mouth, esophagus and stomach.
Effects include: Nausea, pain and vomiting.

SECTION 4 - FIRST AID MEASURES:

Skin Contact: Wash with soap and water.
If discomfort or irritation persists, consult a physician.
Remove contaminated clothing and wash before reuse.
Eye Contact: Flush eyes with water for at least 15 minutes while holding eyelids open.
If discomfort or irritation persists, consult a physician.
Ingestion: Do not induce vomiting.
Never give anything by mouth to an unconscious person.
If discomfort or irritation persists, consult a physician.
Inhalation: If symptoms develop, get fresh air. If symptoms persist, consult a physician.

If breathing has stopped, give artificial respiration then oxygen if needed.

SECTION 5 - FIRE AND EXPLOSION HAZARD DATA

Flash Point: > 200° F (Aqueous system)
Flash Point Method: Estimated
Lower Explosion Limit: Not Available
Upper Explosion Limit: Not Available
Auto-Ignition Temperature: Not Available

NFPA Rating:

Health: 1
Flammability: 1
Reactivity: 0

Extinguishing Media: In case of fire, use water spray, dry chemical, Carbon dioxide or foam.
Special Fire Fighting Procedures: Wear self-contained breathing apparatus and complete personal protective equipment when potential for exposure to vapors or products of combustion exist. Water may be used to cool containers to prevent pressure build-up and possible auto-ignition or explosion. Avoid breathing hazardous vapors or products of combustion. Isolate area and keep unnecessary people away. Prevent run-off from fire control or dilution from entering streams or drinking water supplies.
No special procedures specific to this product.
Unusual Fire and Explosion Hazards: None.

SECTION 6 - ACCIDENTAL RELEASE MEASURES:

Spills/Leaks: Use proper personal protective equipment. Do not flush to sewer or allow spilled product to enter waterways. Keep unnecessary people away.
Contain and/or absorb spill with inert material (i.e. sand, vermiculite) then place in a suitable container.
For large spills, dike area and pump waste material into closed containers for disposal or reclamation.

SECTION 7 - HANDLING AND STORAGE

Precautionary Measures: Avoid contact with eyes, skin and clothing.
Do not take internally.
Practice good personal hygiene to avoid ingestion.
Use only with adequate ventilation.
Wash clothing before reuse.
FOR PROFESSIONAL USE ONLY. KEEP OUT OF CHILDREN'S REACH.

SECTION 8 - EXPOSURE CONTROLS AND PERSONAL PROTECTIVE EQUIPMENT

EXPOSURE GUIDELINES (US)

Ingredient	ACGIH TLV			OSHA PEL			
	TWA	STEL	Ceiling	TWA	STEL	Ceiling	Substance Specific and Mineral Dust PELs

EXPOSURE GUIDELINES (CANADA)

Employers should consult local Provincial regulatory limits for exposure guidelines that may vary locally.

Engineering Controls: Not generally required.

Personal Protective Equipment:

Respiratory Protection: Respiratory protection is not normally required. However, a chemical cartridge respirator with organic vapor cartridge and a pre-filter for dusts/mists is required at or above the applicable exposure limits (Consult above Exposure Guidelines). If no limits exist, use an approved respirator whenever a vapor or mist is generated or if respiratory irritation occurs. Supplied air respirator (SCBA) is required at exposure levels above the capabilities of a chemical cartridge respirator.
Skin Protection: Rubber or other impervious gloves should be worn to prevent skin contact.
Eye Protection: At minimum, safety glasses with side shields should be worn where exposure to excessive dust or spray is likely.
Work/Hygienic Practices: Use good personal hygiene practices.

None beyond those noted above.

SECTION 9 - PHYSICAL AND CHEMICAL PROPERTIES

Physical State:	Liquid
Appearance/Odor:	Blue Liquid
Odor Threshold: (ppm)	Not Determined
pH:	4.8 - 6.8
Vapor Pressure: (Mm Hg)	Unknown
Vapor Density: (Air = 1)	Unknown
Solubility In Water:	Unknown
Specific Gravity: (Water = 1)	1.050-1.070 (expected value)
Evaporation Rate: (Butyl Acetate = 1)	Unknown
Boiling Point:	>212°F/100°C
Viscosity:	Unknown
Bulk Density: (Pounds/Cubic Foot)(Pcf)	Not Applicable
% Volatiles (gr/L): (70°F) (21°C)	~60% (As Water)

SECTION 10 - STABILITY AND REACTIVITY

Chemical Stability:	Stable
Conditions To Avoid:	None known for this product.
Hazardous Polymerization:	Will not polymerize.
Hazardous Decomposition Products:	None known for this product.

SECTION 11 - TOXICOLOGICAL INFORMATION

Ingredient(No data unless listed.) **CAS Number** **LD50 and LC50**

Carcinogenicity:

Ingredient	IARC Group 1	IARC Group 2A	IARC Group 2B	NTP Known	NTP Suspect	OSHA

Mutagenicity:	Not applicable.
Teratogenicity:	Not applicable.
Reproductive Toxicity:	Not applicable.

SECTION 12 - ECOLOGICAL INFORMATION

Environmental Fate:	No data available for product.
Ecotoxicity:	No data available for product.

SECTION 13 - DISPOSAL CONSIDERATIONS

Waste Disposal Procedures: Consult all regulations (federal, state, provincial, local) or a qualified waste disposal firm when characterizing waste for disposal. According to EPA (40 CFR § 261), waste of this product is not defined as hazardous. Dispose of waste in accordance with all applicable regulations.

SECTION 14 - TRANSPORTATION INFORMATION

Proper Shipping Name:	Not Applicable
UN/NA Number:	Not Applicable
Domestic Hazard Class:	Nonhazardous
Surface Freight Classification:	Concrete or Masonry Plasticizer & Water Reducing Compound
Label/Placard Required:	Not Applicable

SECTION 15 - REGULATORY INFORMATION

REGULATORY CHEMICAL LISTS:

CERCLA (Comprehensive Response Compensation and Liability Act):
(None present unless listed below)

Chemical Name	CAS #	Wt %	CERCLA RQ

W. R. GRACE
MATERIAL SAFETY DATA SHEET

Product Name: ADVA Cast 575
MSDS ID Number: D-06201 MSDS Date: 11/14/2006

SARA Title III (Superfund Amendments and Reauthorization Act)

SARA Section 312/Tier I & II Hazard Categories:

Health Immediate (acute)	Yes
Health Delayed (chronic)	No
Flammable	No
Reactive	No
Pressure	No

302 Reportable Ingredients (Identification Threshold 1%.):

Chemical Name	CAS #	Wt %	SARA 302 TPQ

313 Reportable Ingredients (Chemicals present below reporting threshold are exempt):

Chemical Name	CAS #	Wt %
Methyl isothiocyanate	000556-61-6	.0002
Tetrahydro-3,5-dimethyl-2H-1,3,5-thiadiazine-2-thione	000533-74-4	.02

National Volatile Organic Compound Emission Standards For Architectural Coatings:

Volatile Organic Content: (gr/L) Not Applicable

WHMIS Classification(s): D2 B

This product has been classified in accordance with the hazard criteria of the Controlled Products Regulations (CPR). This MSDS contains all the information required by the CPR.

State Regulatory Information:

California Proposition 65: WARNING! This product contains substances known to the state of California to cause cancer, birth defects or other reproductive harm.

Massachusetts Hazardous Substance List(Identification threshold 0.0001%(1ppm)):

Chemical Name	CAS #	Wt %
Methyl isothiocyanate	000556-61-6	.0002

New Jersey Hazardous Substance List (Identification threshold (0.1%)):

Chemical Name	CAS #	Wt %

Pennsylvania Hazardous Substance List (Identification threshold 0.01%):

Chemical Name	CAS #	Wt %

CHEMICAL INVENTORY STATUS:

All chemicals in this product are listed or exempt from listing in the following countries:

US	CANADA		EUROPE	AUSTRALIA	JAPAN	KOREA	PHILIPPINES
TSCA	DSL	NDSL	EINECS/ELINCS	AICS	ENCS	ECL	PICCS
Yes	Yes	No	Not Determined	Not Determined	Not Determined	Not Determined	Not Determined

SECTION 16 - OTHER INFORMATION

Non-Hazardous Ingredient Disclosure:

Chemical Name	CAS Number
Water	007732-18-5
Proprietary Inert	prop00091

Prepared by: EH&S Department
Approved by: EH&S Department
Approved Date: 11/14/2006

Disclaimer:

"The data included herein are presented in accordance with various environment, health and safety regulations. It is the responsibility of a recipient of the data to remain currently informed on chemical hazard information, to design and update its own program and to comply with all national, federal, state and local laws and regulations applicable to safety, occupational health, right-to-know and environmental protection."

APPENDIX C

SECTION 1 - CHEMICAL PRODUCT AND COMPANY IDENTIFICATION

Product Name:	V-MAR 3
MSDS Number:	D-06280
Cancelled MSDS Number:	D-06272
MSDS Date:	07/20/2007
Chemical Family Name:	Dispersed Carbohydrate
Product Use:	Viscosity Modifying Agent for Concrete
Chemical Formula:	Mixture-NA
CAS # (Chemical Abstracts Service Number):	Mixture-NA

Manufactured by:

W.R.Grace & Co.-Conn.
62 Whittemore Avenue
Cambridge, MA 02140

Grace Canada, Inc.
294 Clements Road West
Ajax, Ontario L1S 3C6

In Case of Emergency Call:
In USA: (617) 876-1400 In Canada: (905) 683-8561

SECTION 2 - COMPOSITION/INFORMATION ON INGREDIENTS

Ingredient	CAS#	Percent (max)
Sodium hydroxide	001310-73-2	< 1

SECTION 3 - HAZARDS IDENTIFICATION

Emergency Overview:

Caution!
Causes eye irritation.
Causes skin irritation.

HMIS Rating:

Health:	1
Flammability:	0
Reactivity:	0
Personal Protective Equipment:	B (See Section 8)

Potential Health Effects:

Inhalation: Overexposure is not likely to occur unless specific use generates dust, vapors or mist.
If inhaled as a vapor or mist, causes irritation, sore throat, coughing and breathing difficulty.
Effects include: No other effects expected unless listed below.
Eye Contact: Eye contact causes irritation.
Skin Contact: Skin contact causes irritation.
Skin Absorption: Not expected to be harmful if absorbed through the skin.
Ingestion: Ingestion not expected to be harmful.
Effects include: No other effects expected unless listed below.

SECTION 4 - FIRST AID MEASURES:

Skin Contact: Wash with soap and water.
If discomfort or irritation persists, consult a physician.
Remove contaminated clothing and wash before reuse.
Eye Contact: Flush eyes with water for at least 15 minutes while holding eyelids open.
If discomfort or irritation persists, consult a physician.
Ingestion: Do not induce vomiting.
Never give anything by mouth to an unconscious person.
If discomfort or irritation persists, consult a physician.
Inhalation: If symptoms develop, get fresh air. If symptoms persist, consult a physician.
If breathing has stopped, give artificial respiration then oxygen if needed.

SECTION 5 - FIRE AND EXPLOSION HAZARD DATA

Flash Point:	>200F (Aqueous System)
Flash Point Method:	Not Applicable
Lower Explosion Limit:	Not Available
Upper Explosion Limit:	Not Available
Auto-Ignition Temperature:	Not Available

NFPA Rating:

Health:	1
Flammability:	0
Reactivity:	0

Extinguishing Media: In case of fire, use water spray, dry chemical, Carbon dioxide or foam.
Special Fire Fighting Procedures: Wear self-contained breathing apparatus and complete personal protective equipment when potential for exposure to vapors or products of combustion exist. Water may be used to cool containers to prevent pressure build-up and possible auto-ignition or explosion. Avoid breathing hazardous vapors or products of combustion, keep upwind. Isolate area and keep unnecessary people away. Prevent run-off from fire control or dilution from entering streams or drinking water supplies.
No special procedures specific to this product.
Unusual Fire and Explosion Hazards: None unless noted below.

SECTION 6 - ACCIDENTAL RELEASE MEASURES:

Spills/Leaks: Use proper personal protective equipment. Do not flush to sewer or allow to enter waterways. Keep unnecessary people away.
Contain and/or absorb spill with inert material (i.e. sand, vermiculite) then place in a suitable container. For large spills, dike area and pump waste material into closed containers for disposal or reclamation.

SECTION 7 - HANDLING AND STORAGE

Precautionary Measures: Avoid contact with eyes, skin and clothing.
Do not take internally.
Practice good personal hygiene to avoid ingestion.
Use only with adequate ventilation.
Wash clothing before reuse.
FOR PROFESSIONAL USE ONLY. KEEP OUT OF CHILDREN'S REACH.

SECTION 8 - EXPOSURE CONTROLS AND PERSONAL PROTECTIVE EQUIPMENT

EXPOSURE GUIDELINES (US)

Ingredient	ACGIH TLV			OSHA PEL			
	TWA	STEL	Ceiling	TWA	STEL	Ceiling	Substance Specific and Mineral Dust PELs
Sodium hydroxide	-	-	2 mg/m3 Ceiling	2 mg/m3 TWA	-	-	-

EXPOSURE GUIDELINES (CANADA)

Employers should consult local Provincial regulatory limits for exposure guidelines which may vary locally.

Engineering Controls: Not generally required.

Personal Protective Equipment:

Respiratory Protection: Respiratory protection is not normally required. However, a chemical cartridge respirator with organic vapor cartridge and a prefilter for dusts/mists is required at or above the applicable exposure limits (Consult above Exposure Guidelines). If no limits exist, use an approved respirator whenever a vapor or mist is generated or if respiratory irritation occurs. Supplied air respirator (SCBA) is required at exposure levels above the capabilities of a chemical cartridge respirator.
Skin Protection: Rubber or other impervious gloves should be worn to prevent skin contact.

Eye Protection: At minimum, safety glasses with side shields should be worn where exposure to excessive dust or spray is likely.
Work/Hygienic Practices: Use good personal hygiene practices.
None beyond those noted above.

SECTION 9 - PHYSICAL AND CHEMICAL PROPERTIES

Physical State:	Liquid
Appearance/Odor:	Thick White Paste
Odor Threshold: (ppm)	Not Determined
pH:	Not Available
Vapor Pressure: (Mm Hg)	Not Available
Vapor Density: (Air = 1)	Not Available
Solubility In Water:	Miscible
Specific Gravity: (Water = 1)	1.0
Evaporation Rate: (Butyl Acetate = 1)	Unknown
Boiling Point:	>212°F/100°C
Viscosity:	Unknown
Bulk Density: (Pounds/Cubic Foot)(Pcf)	Not Applicable
% Volatiles (gr/L): (70°F) (21°C)	~80% (As Water)

SECTION 10 - STABILITY AND REACTIVITY

Chemical Stability:	Stable
Conditions To Avoid:	None known for this product.
Hazardous Polymerization:	Will not polymerize.
Hazardous Decomposition Products:	None known for this product.

SECTION 11 - TOXICOLOGICAL INFORMATION

Ingredient(No data unless listed.) **CAS Number** **LD50 and LC50**

Carcinogenicity:

Ingredient	IARC Group 1	IARC Group 2A	IARC Group 2B	NTP Known	NTP Suspect	OSHA

Mutagenicity:	Not applicable.
Teratogenicity:	Not applicable.
Reproductive Toxicity:	Not applicable.

SECTION 12 - ECOLOGICAL INFORMATION

Environmental Fate:	No data available for product.
Ecotoxicity:	No data available for product.

SECTION 13 - DISPOSAL CONSIDERATIONS

Waste Disposal Procedures: Consult all regulations (federal, state, provincial, local) or a qualified waste disposal firm when characterizing waste for disposal. According to EPA (40 CFR § 261), waste of this product is not defined as hazardous. Dispose of waste in accordance with all applicable regulations.

SECTION 14 - TRANSPORTATION INFORMATION

Proper Shipping Name:	Not Applicable
UN/NA Number:	Not Applicable
Domestic Hazard Class:	Nonhazardous
Surface Freight Classification:	Not Applicable
Label/Placard Required:	Not Applicable

SECTION 15 - REGULATORY INFORMATION

REGULATORY CHEMICAL LISTS:

CERCLA (Comprehensive Response Compensation and Liability Act):
(None present unless listed below)

Chemical Name	CAS #	Wt %	CERCLA RQ

SARA Title III (Superfund Amendments and Reauthorization Act)

SARA Section 312/Tier I & II Hazard Categories:

Health Immediate (acute)	No
Health Delayed (chronic)	No
Flammable	No
Reactive	No
Pressure	No

302 Reportable Ingredients (Identification Threshold 1%.):

Chemical Name	CAS #	Wt %	SARA 302 TPQ

313 Reportable Ingredients (Chemicals present below reporting threshold are exempt):

Chemical Name	CAS #	Wt %
Methyl alcohol	000067-56-1	.0564
Tetrahydro-3,5-dimethyl-2H-1,3,5-thiadiazine-2-thione	000533-74-4	.3384

National Volatile Organic Compound Emission Standards For Architectural Coatings:

Volatile Organic Content: (gr/L) 0

WHMIS Classification(s): Not Controlled

This product has been classified in accordance with the hazard criteria of the Controlled Products Regulations (CPR). This MSDS contains all the information required by the CPR.

State Regulatory Information:

California Proposition 65: WARNING! This product contains substances known to the state of California to cause cancer, birth defects or other reproductive harm.

Massachusetts Hazardous Substance List(Identification threshold 0.0001%(1ppm)):

Chemical Name	CAS #	Wt %

New Jersey Hazardous Substance List(Identification threshold (0.1%)):

Chemical Name	CAS #	Wt %
Tetrahydro-3,5-dimethyl-2H-1,3,5-thiadiazine-2-thione	000533-74-4	.3384

Pennsylvania Hazardous Substance List(Identification threshold 0.01%):

Chemical Name	CAS #	Wt %

CHEMICAL INVENTORY STATUS:

All chemicals in this product are listed or exempt from listing in the following countries:

US	CANADA		EUROPE	AUSTRALIA	JAPAN	KOREA	PHILIPPINES
TSCA	DSL	NDSL	EINECS/ELINCS	AICS	ENCS	ECL	PICCS
Yes	Yes	No	No	No	No	No	No

SECTION 16 - OTHER INFORMATION

Non-Hazardous Ingredient Disclosure:

Chemical Name	CAS Number
Water	007732-18-5
s-657	125005-87-0

Prepared by: EH&S Department
Approved by: EH&S Department
Approved Date: 07/20/2007

Disclaimer:

"The data included herein are presented in accordance with various environment, health and safety regulations. It is the responsibility of a recipient of the data to remain currently informed on chemical hazard information, to design and update its own program and to comply with all national, federal, state and local laws and regulations applicable to safety, occupational health, right-to-know and environmental protection."

APPENDIX D

NASP STRAND BOND TEST (DRAFT)

Standard Test Method to Assess the Bond of 0.5 in. (12.7 mm) Seven Wire Strand with Cementitious Materials

1. **Scope**

   1.1 This test method provides a means to assess the ability of 0.5 in. (12.7 mm) seven wire strand to bond with concrete and other cementitious products. The method tests the bondability of strands that are made and intended for use as prestressing strands that conform to ASTM A 416.

   1.2 This test does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the use of this test method to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. **Reference Documents**

   2.1 ASTM A 416
   2.2 ASTM C 33
   2.3 ASTM C 150
   2.4 ASTM C 192

3. **Summary of the Test Method**

   Test specimens are prepared by casting a single, 0.5 in. (12.7 mm) seven wire strand into a cylinder of concrete mortar with a bonded length of 16 in. (400 mm). The constituents and proportions for the concrete mortar mixture are prescribed. The concrete in the specimen is cured for approximately one day under controlled conditions. The specimen is tested at one day of age by pulling the strand through the mortar at a prescribed rate of loading. The pull-out force is recorded at 0.10 in. (2.5 mm) of total slip. A single NASP Bond Test shall consist of 6 or more individual pull-out tests. The strand for the NASP Bond Test shall be taken from the same lot or reel of strand.

4. **Preparation of Test Specimens**

   4.1 Strand Specimens. The strand shall conform to ASTM A 416 and shall be intended for use in pretensioned or post-tensioned applications. Strand specimens for a single NASP Strand Bond Test shall be taken from the same lot or the same reel of prestressing strand. A minimum of six strand specimens are required for a single NASP Strand Bond Test.

   4.1 Concrete Mortar Mixture Constituents and Proportions. The concrete mortar mixture shall consist of sand, cement and water mixed thoroughly in the following proportions: 2 parts sand, 1 part cement and 0.45 parts water. The sand shall conform to ASTM C 33 requirements for **Fine Aggregate.** The batch weight for sand shall be computed using the aggregate's unit weight at saturated surface dry (SSD) conditions. In computing weights for mixture proportions, the moisture content within the sand shall be accurately sampled and measured. The mixture proportions shall be corrected for the moisture content measured in the sand prior to mixing. Batch materials shall be handled in conformance with ASTM C 192. The cement shall conform to ASTM C 150 requirements for Type III cement. The water shall be potable and suitable for making concrete.

4.2 Mixing. The concrete mortar and the test specimens shall be made in conformance with ASTM C 192. Measurements of slump and air content are not required.

4.2 Curing. The concrete mortar and test specimens shall be cured in conformance with ASTM C 192. The concrete mortar shall be cured at 73 ± 3•F (23 ± 2•C) from the time of molding until the moment of test. Storage during the curing period shall be in a vibration-free environment.

4.3 Mortar Strength. Concrete mortar strength shall be evaluated in conformance with ASTM C 109 using 2 in. (51 mm) mortar cubes, except that the mixture proportions for the mortar are given in Section 4.1 and flow measurement shall not be required. The average mortar cube strength at the time of the NASP Bond Test shall not be less than 3500 psi (500 kPa). Mortar cube strength shall not exceed 5000 psi (700 kPa) at the time of the NASP test.

4.4 Test specimens shall be made by casting one single strand concentrically in concrete mortar within a 5 in. (125 mm) diameter steel casing as described in Fig. 1. The length of the steel tube shall be 18 in. as shown. The bonded length of the strand shall be 16 in., with a 2 in. long bond breaker as shown in the figure. The steel casing shall have sufficient rigidity to prevent radial cracking in the specimen during testing. The test specimen shall be cast with the longitudinal axis of the strand and the steel casing in the vertical position. Test specimens shall be mechanically consolidated by vibration in conformance with ASTM C 192.

5. **Test Procedure.**

5.1 Timing of the Test. The NASP Bond Test shall be conducted 24 ± 2 hrs. from the time of casting the specimens.

5.2 Instrumentation and measurement. The pull-out force shall be measured by a calibrated load measuring device, either electronically or hydraulically, or in combination of hydraulics and electronics. Pull-out force shall be measured to the nearest 10 lb increments. The relative movement of the strand to the hardened concrete mortar shall be measured. This measurement is typically called the "free-end slip" and shall be measured to 0.01 in. The slip shall be measured by a calibrated device.

5.3 Strand shall be pulled from the concrete by reacting against the transverse steel plate. The loading shall be controlled by strand displacement measured at the point where the load is applied to the strand. The displacement rate shall be 0.1 in. per minute (2.5 mm per minute).

5.4 The strand shall be loaded at a distance approximately 6 in. from the end of the specimen.

5.5 The pull-out force shall be recorded when the opposite end of the strand, or the "free end" achieves a total displacement of 0.10 in. relative to the hardened concrete mortar.

5.6 If the hardened concrete mortar exhibits cracking in two or more of the six individual tests, then all results of NASP Strand Bond Test shall be discarded and new specimens prepared for a new NASP Strand Bond Test.

6. Reporting.

6.1 Sample Size. A single NASP Strand Bond Test shall consist of a minimum of six (6) individual tests conducted on single strand specimens.

6.2 For each individual test, report the pull-out force that corresponds to a relative displacement of 0.10 in. between the strand and the hardened concrete mortar.

6.3 For the NASP Bond Test, compute the average pull-out force from the individual tests and report the value as the average value for the NASP Bond Test. If one of the specimens exhibited radial cracking during testing, disregard the pull-out value of that specimen when reporting results. If two or more of the specimens exhibit radial cracking, the entire results should be disregarded and the NASP Bond Test performed again in its entirety.

7. Acceptance

7.1 The strand shall be accepted for pretensioned and post-tensioned prestressed applications when the average value of the NASP Strand Bond Test is not less than __________ lbs and no individual test result is less than __________lbs.

DRAFT
Bruce W. Russell, Ph.D., P.E.
August 30, 2001

VITA

Saugata Purkait

Candidate for the Degree of

Master of Science

Thesis: ASSESSING THE BOND QUALITY OF PRESTRESSING STRANDS IN SCC USING NASP BOND TEST.

Major Field: Civil Engineering.

Biographical:

Personal Data: Born in Kolkata, West Bengal, India, On September 10, the son of Mrityunjoy Purkait and Pranati Purkait.

Education: Graduated from Kendriya Vidyalaya Ballygunge, Kolkata, India in May 2000; received Bachelor of Engineering in Civil Engineering from University of Pune, India, 2005. Completed the requirements for the Master of Science degree with Major in Civil Engineering with Structural Engineering as Speciality at Oklahoma State University in Dec ember, 2008.

Experience: Employed at Vicon Infrastructure Pvt Ltd, Raipur India as a Site Engineer in Summer 2004; employed in Vicon Infrastructure Pvt.Ltd as a Structural Design Engineer Trainee in Aug 2005 – May 2006; Employed by Oklahoma State University as a Graduate Research Associate and graduate teaching assistant 2006 to present.

Professional Memberships: American Concrete Institute, Indian Institure of Engineers, Prestressed Concrete Institute, Portland Cement Association, American Society of Civil Engineers, Structural Engineering Institute.

Name: Saugata Purkait Date of Degree: December, 2008

Institution: Oklahoma State University Location: Stillwater, Oklahoma

Title of Study: ASSESSING THE BOND QAULITY OF PRESTRESSING STRANDS WITH SCC BY USING NASP BOND TEST.

Pages in Study: 211 Candidate for the Degree of Master of Science

Major Field: Civil Engineering

Scope and Method of Study:

This thesis highlights effects of SCC on the bond performance of pre-stressing strands. The Standard Test for Bond (also called the North American Strand Producers Bond Test) was performed to evaluate the effect of 0.5 in strand with various SCC mixtures. Research variables include coarse aggregate content and sizes with a fixed rate of HRWRA and VMA. From these test, the pull out values are used to determine whether SCC causes a change in the bond strength of pre-stressing strand. With this, the effects of SCC proportions on the bond properties can be evaluated.

Findings and Conclusions:

The test method showed strong correlation between the Mortar and the SCC bond strength values both with mortar strength and concrete strength. This indicated that the test is a reliable as a standardized bond test. The statistical analysis showed that the mix design had a strong impact with the SCC bond strength. In order to balance the better fresh properties and the higher bond strength, the key is to work on different types of fines as a substitute for coarse aggregates with the HRWRA and VMA in different SCC mixes. SCC behaves very much the same as Normal Concrete in terms of Bond Strength.

ADVISER'S APPROVAL: Dr. Bruce .W. Russell